PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
NOVEMBER 1954

JOINT MEETING WITH THE INSTITUTION OF ELECTRICAL ENGINEERS

25 May, 1954

HAROLD BISHOP, C.B.E., B.Sc., President of the Institution of Electrical Engineers, in the Chair

The Council (of the Institution of Civil Engineers) reported that they had recently transferred to the class of

Members

BAILEY, ARNOLD.
BELL, WILLIAM LESLIE.
FOX, FRANK HENRY GEORGE, B.Sc. (*Glasgow*).
LEE, FRANK.
OSBORNE, FRANCIS VICTOR.
ROSSITER, STANLEY ALBERT, B.Sc.Tech. (*Manchester*).
SMITH, LAWRENCE GILLING.
SUKIAS, BARDICK, B.Sc.(Eng.) (*London*).
WARREN, LESLIE REGINALD, B.Sc. (*Birmingham*).
WILSON, PAUL NORMAN, D.S.C., M.A. (*Cantab.*).
WOODROW, FREDERICK HENRY, O.B.E., B.Sc.(Eng.) (*London*).
YOUNG, HARVEY BRASSINTON.

and had admitted as

Graduates

ABBOTT, MICHAEL BARRY, B.Sc.(Eng.) (*London*).
ANDRÉ, JOHN FREDERICK WILLIAM, Stud.I.C.E.
ARMES, DEREK CHARLES, B.Sc.(Eng.) (*London*), Stud.I.C.E.
BARCLAY, GEORGE ELLIS, B.Sc. (*Glasgow*), Stud.I.C.E.
BARNETT, CLIFFORD RAYMOND, B.E. (*New Zealand*), Stud.I.C.E.
BRADSTOCK, JOHN WALTER, B.E. (*New Zealand*), Stud.I.C.E.
BROWN, ROBERT HENRY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
BYERS, ALEXANDER IAN LAIDLAW, B.Sc. (*Glasgow*).
CAMERON, JAMES DUNCAN, Stud.I.C.E.
CANNON, DAVID EDWIN, B.A. (*Cantab.*).
COHEN, MAURICE, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Connor, David Young, Stud.I.C.E.
Cumberland, Norman Eldrid.
Currie, Alastair Thomas, Stud.I.C.E.
Dalgarno, Charles William, B.Sc. (*Aberdeen*).
Davidson, James Young, B.Sc. (*St Andrews*).
Easterbrook, Colin Drake, B.Sc. (*Wales*).
Evans, David Edmund, Stud.I.C.E.
Everett, John William, B.Sc. (*Edinburgh*), Stud.I.C.E.
Finnegan, John William, B.E. (*National*).
Gawn, John Andrew, B.Sc. (*Belfast*), Stud.I.C.E.
Ghalayini, Ahmad Rashid.
Gittins, Roy William, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hallows, Brian Hector, Stud.I.C.E.
Hibbert, Harry, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Hillier, Derek John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hooper, Patrick Norman, B.Sc. (*Glasgow*), Stud.I.C.E.
Johnson, Keith Maxwell, B.Sc. (*Glasgow*), Stud.I.C.E.
Lakeman, Brian George Humphrey, B.Sc. (*Wales*), Stud.I.C.E.
Lewis, Hugh Elvet, B.Sc. (*Wales*).
Lovell, John Sydney, B.Sc.(Eng.) (*London*), Stud.I.C.E.
MacBean, Ian Gordon Gillies, Stud. I.C.E.
Mackay, Roderick Duncan, B.Sc. (*Aberdeen*).
Marshall, William Eric, B.Sc.(Eng.) (*London*).
Martin, Simon Fraser, B.Sc.(*Natal*).
Metelerkamp, David Rawstorne.
Moffat, Peter Alec Spencer, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Monk, Richard Burton, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Nicol, Raymond Sydney George, B.Sc. (*Edinburgh*), Stud.I.C.E.
Olney, Keith Philip, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Ponsonby, Thomas, Stud.I.C.E.
Poole, Clifford Jerrold, Stud.I.C.E.
Reid, Roland Cyril Ernest, B.Sc. (Eng.) (*London*).
Rennie, David McCall, B.A. (*Cantab.*).
Russell, Harold Victor, B.Sc. (*Belfast*).
Severn, Brian, B.A. (*Oxon.*), Stud.I.C.E.
Sharman, Richard Harry, B.Sc. (*Nottingham*).
Thompson, Christopher Ward, Stud. I.C.E.
Turrell, Harold Peter, Stud.I.C.E.
Weir, Charles Edwin, B.Sc. (*Belfast*).
Whitfield, Philip Douglas, B.Sc. (Eng.) (*London*).
Wilson, Norman William.
Wilson, William Jack Craig, Stud. I.C.E.
Wilson, William Notman, M.A. (*Cantab.*).

and had admitted as

Students

Andrews, Vernon Hugh.
Batty, Robert Alfred.
Bell, William Patrick Leslie.
Betteley, John Dennis.
Bhalla, Dharm Pal Singh.
Boddy, James Edward.
Bottomley, David Leslie.
Breakell, Richard Farran.
Burch, Brian Thomas.
Burke, Raymond Owen.
Callard, Anthony John.
Cave, Thomas Geoffrey.
Costelloe, Michael Joseph.
Dunn, Nicholas Howard.
Dutta-Munsi, Tushar Kanti.
Edmans, Paul Robin.
Ellison, Eric.
Fogg, Brian.
Gilhooley, James.
Glithero, Gordon Arnold.
Gooding, Barrie.
Grant, Victor Thomas.
Griffiths, James Peter.
Hodgett, Graham.
Holcroft, Cyril.
Hutton, Herbert Robinson.
Jesper, David Guy Sinclair.
John, Alan Edward.
Ledingham, Leslie William Clark.
McCulloch, Alexander Griffith
McPhail, Norman Arthur.
McShane, John.
Manson, Erik John.
Neville, Donald Joseph.
Paulger, Terence Walter.
Perry, Leonard Ernest Arthur.
Powrie, David Eustace.
Rance, Peter John.
Ravenscroft, William.
Rendall, Robert Peterson.
Sandhu, Balbir Singh.
Sankey, Ronald Brian.

Shore, Malcolm Robert.
Simperingham, Patrick.
Steedman, James Cyril.
Tailby, Ian George William.
Walker, Michael Bryan.
Whittle, Ian Richardson.
Wilkinson, Robin Anthony.
Wood, John Douglas.
Young, Robert John.

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Institutions were accorded to the Authors.

Paper No. 6007

"Owen Falls, Uganda, Hydro-Electric Development"

by

Sir Charles Redvers Westlake, M.I.E.E.,

Reginald William Mountain, B.Sc.(Eng.), M.I.C.E., M.I.E.E.,

and

Thomas Angus Lyall Paton, B.Sc.(Eng.), M.I.C.E.

SYNOPSIS

The Paper describes the hydro-electric development at Owen Falls and the economics of the scheme. The development is under the control of the Uganda Electricity Board, formed in 1948.

Owen Falls is on the White Nile about 2 miles downstream from the sole outlet from Lake Victoria, the third largest lake in the World, 26,000 square miles in area. A mass-concrete gravity dam, 100 feet high and 2,500 feet long, has been built with a power station alongside for an ultimate plant installation of ten Kaplan turbines, each coupled to a 15,000-kilowatt alternator. One unit commenced generating in January 1954, and five more units are being installed progressively.

In addition to power generation, the scheme is designed to provide long-term storage in Lake Victoria for the benefit of Egypt.

The average rainfall on the catchment area of 103,000 square miles is 43 inches but only 7 per cent of the run-off flows down the White Nile. Despite this, the volume of water available at Owen Falls is considerable, ranging from the minimum of 10,500 cusecs to the maximum of 43,000.

The capital cost of the scheme, excluding transmission, is about £12 million. The revenue of the Uganda Electricity Board has risen from £70,000 a year in 1948 when the maximum demand was 1½ megawatt to an anticipated £700,000 a year in 1954. It is estimated that demand will reach 90 megawatts in 1962. Maximum output is 700 million units per annum with 60-per-cent load-factor.

INTRODUCTION

The first mention in writing of a possibility of utilizing the power available from the River Nile, in Uganda, for industrial development was made in 1904 by what is now known as The Uganda Company. The Company suggested that the Ripon Falls, which constituted the outfall of Lake Victoria, might be harnessed for power production for industrial development.

In 1907, the Right Honourable (now Sir) Winston Churchill visited Uganda as Under-Secretary of State for the Colonies and in his book, "My African Journey" wrote as follows :—

> "We must have spent three hours watching the waters and revolving plans to harness and bridle them. So much power running

> to waste, such a coign of vantage unoccupied, such a lever to control the natural forces of Africa ungripped, cannot but vex and stimulate imagination. And what fun to make the immemorial Nile begin its journey by diving through a turbine ! ”

In 1935, the East African Governments commissioned two eminent firms of consulting engineers to report on the utilization of the water-power resources of the Victoria Nile. Unfortunately at that time it did not appear to the consulting engineers that there was adequate demand for the power to justify what was then thought to be a costly scheme.

In 1947, the Government of Uganda called for a report on the potential demand which might be expected within a reasonable distance of the Owen Falls. As a result, the Westlake Report of 1947 was prepared and later unanimously adopted by the Uganda Legislative Council.

In January 1948, the Uganda Electricity Board was set up and by September of that year most of the plant had been placed on order.

Concurrently the Board was dealing with the installation of 15,000 kilowatts of thermal plant. The function of this plant was to fill the gap between 1948 and the commissioning of the Hydro-electric Scheme in 1954, and during this period every endeavour was to be made to build up a substantial load to be taken over by the hydro-electric power station. At the same time, the Board placed extensive contracts for the erection of transmission and distribution lines.

In September 1949, the major contract for the civil engineering works was placed and work commenced upon the site very shortly afterwards.

To save time, and well before the placing of the civil engineering contract, the Board had constructed an African camp near the site of the works to house 1,200 Africans, and on the other bank of the river a £400,000 housing scheme was embarked upon to house the European artisans and staff who would be required for the work. This advanced planning was of the utmost assistance to the contractor and reduced delays in the commencement of the work.

The first concrete was laid in May 1951, and since that date work has gone on without a break and without labour troubles, and the commissioning of the first generator was accomplished on almost the scheduled date.

Water Power Potential of the White Nile

The hydrological data for the design of the Owen Falls scheme have fortunately been recorded over a long period. Continuous readings of the lake level have been kept since 1896 by the Physical Department of the Ministry of Public Works, Egypt, who also confirmed that the flow in the River is directly related to the lake level.

Lake Victoria lies on a plateau at an elevation of about 3,716 feet above sea level. It has a water-surface area of 26,000 square miles and is surrounded by low mountain ranges up to 9,000 feet in height, of which the highest peaks lie to the north-east.

The average rainfall in the catchment area of 103,000 square miles is in the region of 43 inches per annum. The wettest area is centred around the Sesse Islands, off the north-west shores and the driest area is on the southern shores of the lake, the average annual intensities being 87 inches and 30 inches respectively.

In view of the very large surface area in relation to the catchment, Lake Victoria regulates the flow of the Nile within close limits, the ratio of maximum to minimum being 4 to 1. Full regulation could be obtained with a 2-metre range in lake level. The average flow of the Nile at the outlet was 22,100 cusecs over the last 50 years, the maximum flow was 43,000 cusecs, and the minimum 10,500 cusecs.

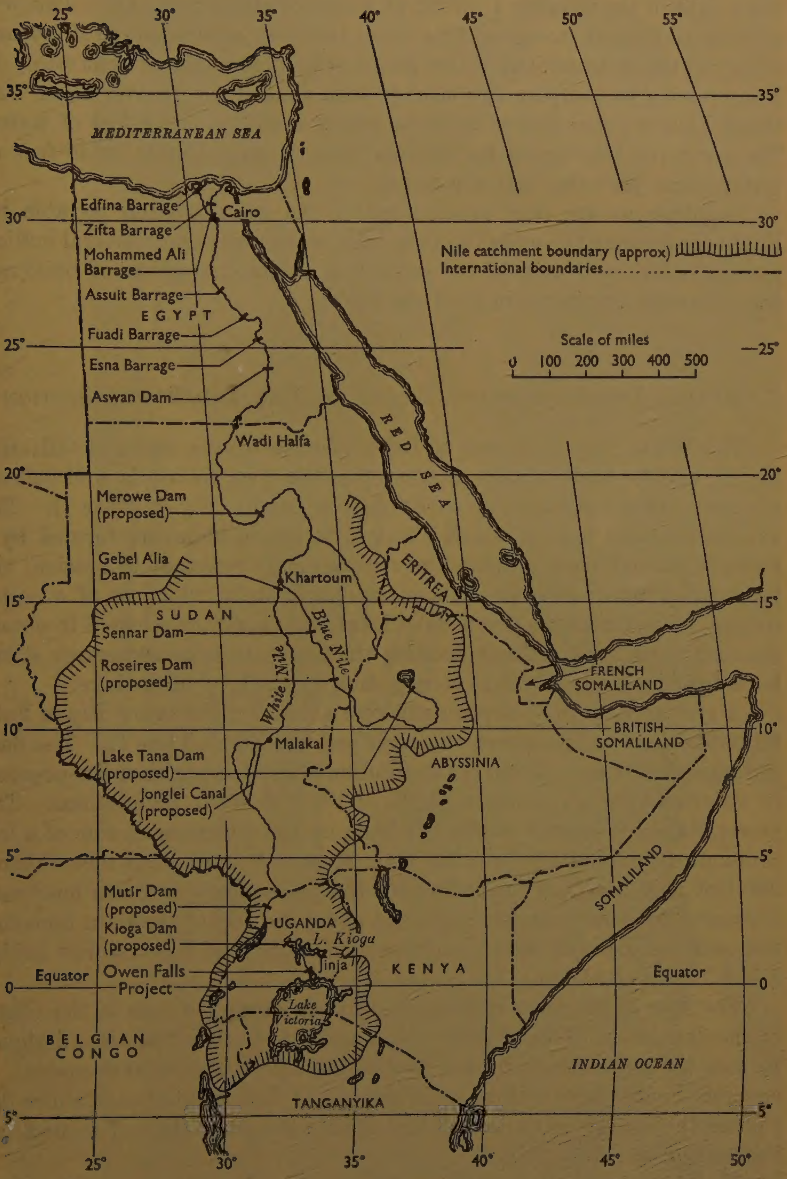
On the other hand, this flow represents only a very small proportion of the total rainfall falling on the catchment area. The estimated average annual loss from the combined land and water area has been calculated to be 41 inches, or 93·3 per cent of the estimated average annual rainfall. The vast lake area coupled with practically 100 per cent vegetation cover over the catchment area has intensified the very high losses caused by evaporation and transpiration.

These losses will, no doubt, be reduced in time by the drainage of swamps formed in most of the river deltas. The water saved will be most valuable in increasing the agricultural production of the East African Territories and of the Nile Valley downstream. But in the meantime Egypt is anxious to build up a reserve of stored water to make up deficiencies in the Lower Nile.

Studies have been made by the Egyptians of several long-term proposals for storage and conservation of water which concern Uganda and particularly the generation of hydro-electric power in the Protectorate (see *Fig. 1*). Lake Victoria, Lake Kioga, and Lake Albert are all included in the overall conservation plan and a dam is also proposed downstream of Lake Albert to balance the flow of certain tributaries. Between Lakes Victoria and Albert, a distance of 234 miles, the Nile falls a total of 1,700 feet, first in a series of falls and rapids over 45 miles to Lake Kioga, and then the principal drop at Murchison Falls, about 9 miles above Lake Albert. It is estimated that the total power potential between Lakes Victoria and Albert is over 2 million kilowatts.

This long-term or " century " storage entails some restriction in the average annual flow of the river which would normally be available for power generation. On the Owen Falls scheme, agreement has been reached to restrict the water available from an average of 22,100 cusecs to 17,700 cusecs, the balance being used to build up storage in the lake until required by Egypt. Compensation will be paid for the loss of power not only at Owen Falls, but for the other water-power schemes down the Nile which may ultimately be developed in Uganda.

Fig. 1



River Nile Basin

Without additional capacity in the lake, full advantage cannot be taken of this balance, and Egypt has, therefore, agreed to pay for increasing the height of the dam by 1 metre, at a cost of about £200,000. This will provide additional storage of 55 million acre-feet, equivalent to 127 years' supply of the Metropolitan Water Board of London at the present rate. It is interesting to compare this storage with that of the Hoover Dam. At Owen Falls 1 cubic foot of concrete stores 1 million cubic feet of water. The corresponding figure for Hoover Dam is only 15,000 cubic feet of water stored per cubic foot of concrete.

Negotiations are now proceeding for the compensation payable by Egypt for flooding the margins of the lake, estimated to cost several million pounds, which will include the protection or reconstruction of buildings and of marine structures in the Lake Ports.

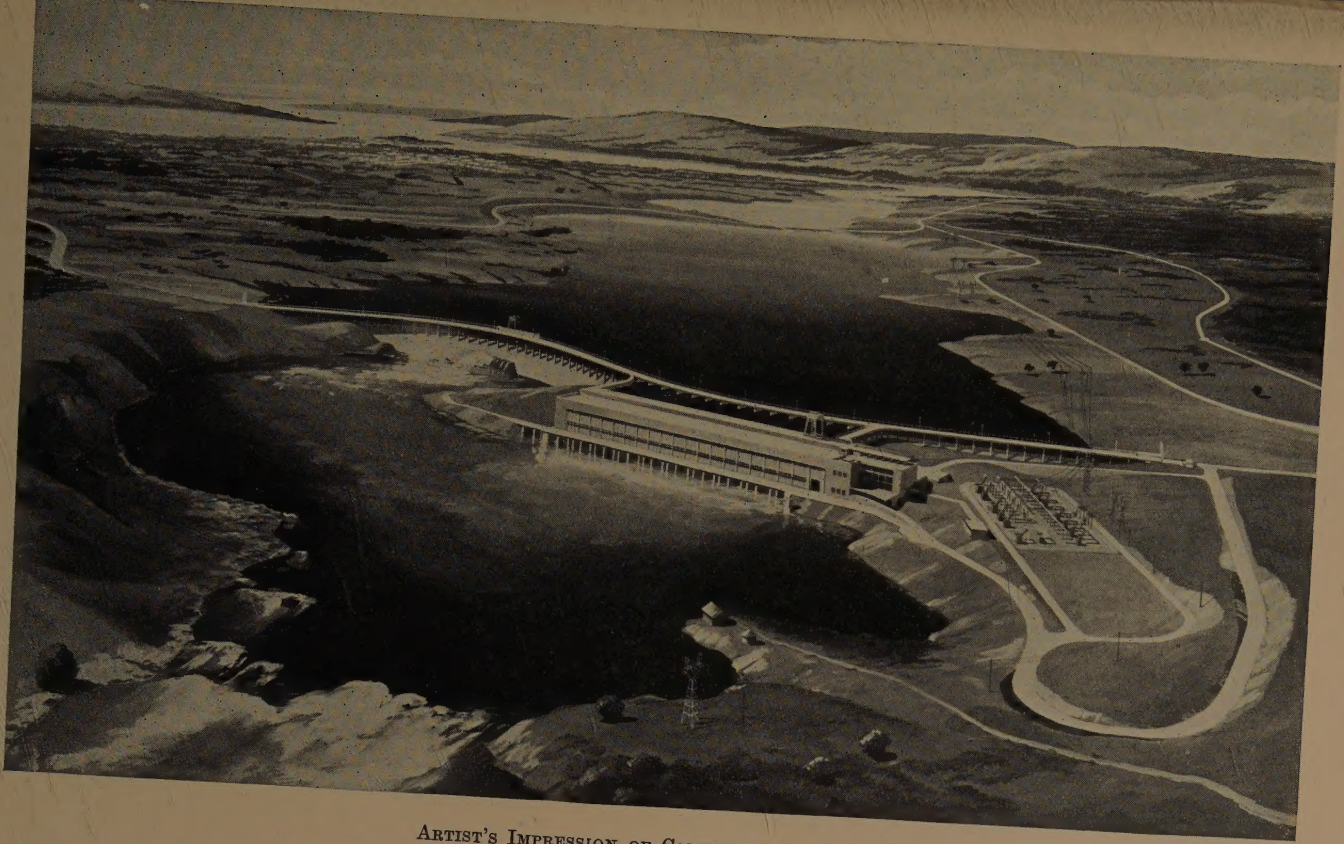
General Description and Design of Dam and Power Station

The White Nile flows out of Lake Victoria over a series of falls and rapids, and for several miles downstream the river follows a well defined channel with steep banks up to 100 feet in height (Fig. 2, Plate 1). The various falls, the first of which is known as Ripon Falls, are formed by a series of bars of hard amphibolite rock strongly resistant to erosion, the intervening material between these bars being shale, which is of a softer nature. The site of the Owen Falls Dam was, therefore, selected to obtain the maximum head without flooding the banks, to take advantage of the hard amphibolite bars and to control directly the level of the lake.

A gravity dam in cyclopean concrete about 2 miles below Ripon Falls forms the main barrier across the river-bed (see Fig. 3, Plate 1). The dam is designed with a slight curve (to a radius of 745 feet) against the pressure of water and has a maximum height of 100 feet above foundations. The power station is located on the left bank on the downstream side of a low promontory jutting out into the main river. A short head-race channel 200 feet long has been formed across the promontory and this head-race is crossed by a road-bridge continued along the top of the dam replacing the existing roadway which is suspended under the Nile railway bridge about 1 mile upstream.

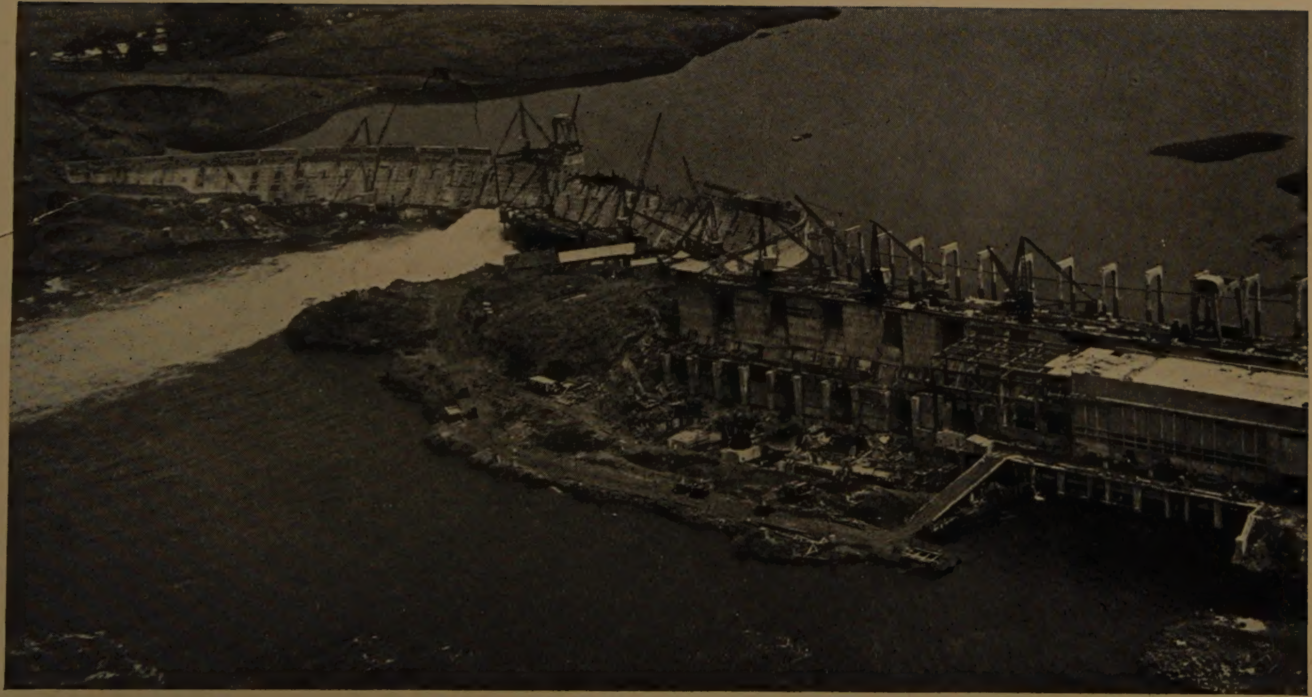
The flow of the river is controlled by six ground sluices in the centre of the dam, each 16 feet 9 inches high and 9 feet 10 inches wide, designed to pass the maximum recorded flow of the Nile, about 43,000 cusecs. A concrete bucket with a slope of 35 degrees at the outlet of each sluice dissipates the energy of the water and avoids the possibility of scour along the back of the dam (*Fig. 5*).

Model experiments were carried out under the direction of Professor C. M. White at Imperial College, South Kensington, to determine the best design and position of the ground sluices and to calibrate the flow. A



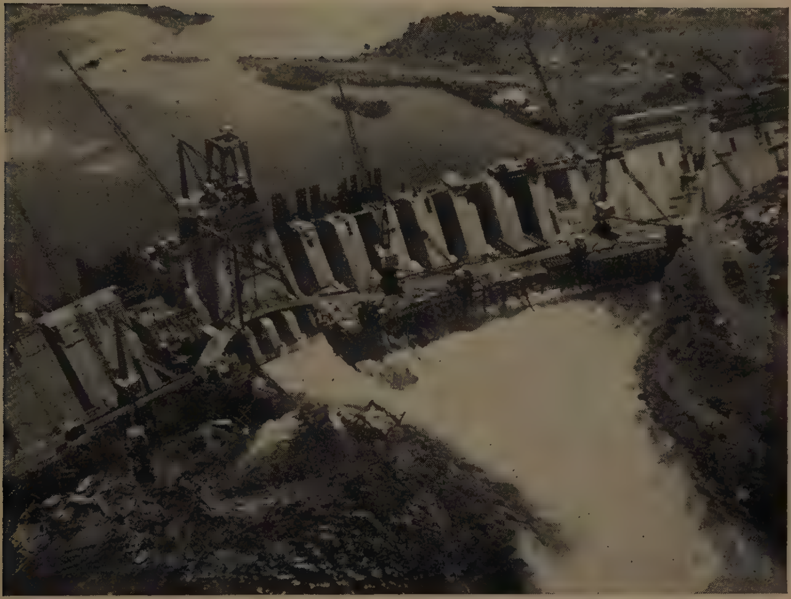
Artist's Impression of Completed Scheme

Fig. 11



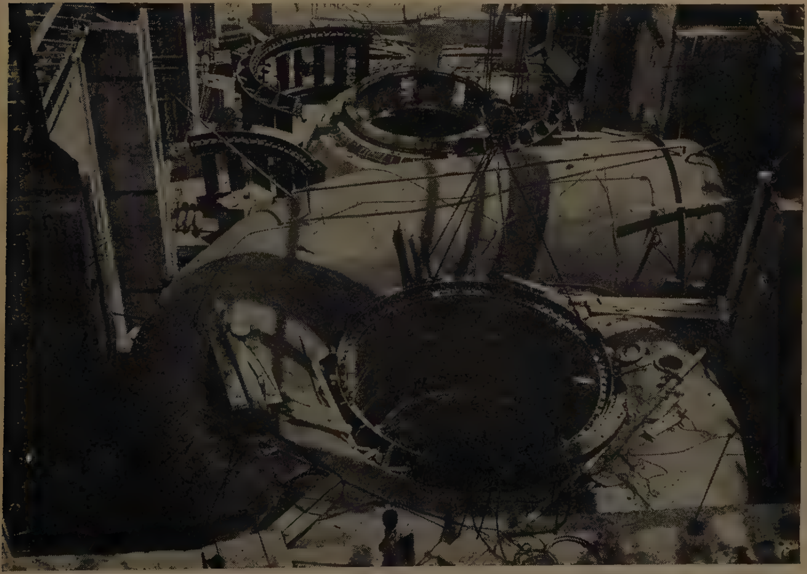
View from Downstream after Partial Flooding. The Main Dam is on the Left and the Power House is on the Right

Fig. 12



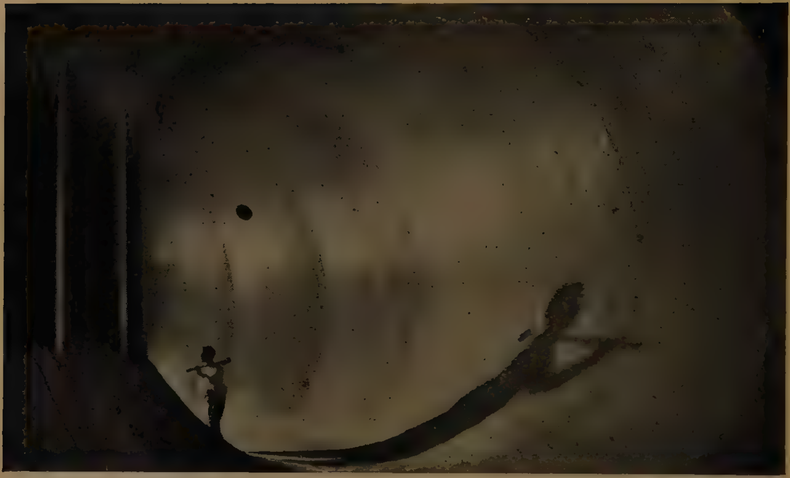
View of Main Dam from Downstream, showing Sluices 1 and 2 in operation under a Low Head and the Temporary Opening in the Dam itself

Fig. 13



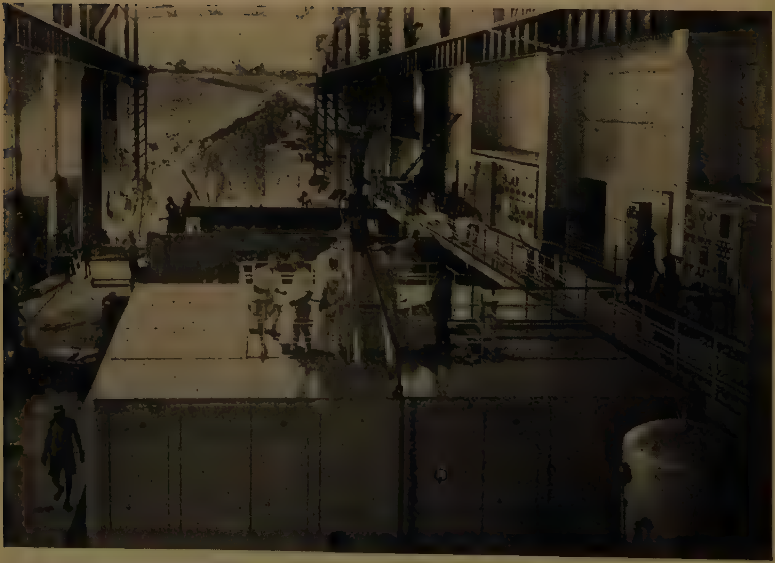
Spiral Casing for Set No. 1 under Construction

Fig. 14



Interior of Spiral Casing for Set No. 1

Fig. 15



View of Power House from Lower Visitor's Gallery, showing Set No. 1 completed summary of these experiments is included in Appendix I. The same facilities were later used by the contractors for the cofferdam design.

Some doubt may be expressed as to whether flood discharge arrangements will prove adequate in the future. Not only has hydrological data been recorded over a very long period but the balancing effect of the lake is very considerable. A flow of 43,000 cusecs would take 6 months to raise the lake by 1 foot. Several years' warning is, therefore, available and provision has been made on the right bank by sterilizing land from building should an additional spillway be required.

In addition to the model tests on the sluices, further experiments were carried out on the design of the turbine intakes by Dr E. F. Gibbs at Bristol University. A summary of the results is given in Appendix II and entailed some modifications to the original design. Each intake is now equipped with one 21-foot-by-23-foot free-roller emergency-gate together with two screens, the grooves for which can accommodate bulkhead gates required for maintenance purposes.

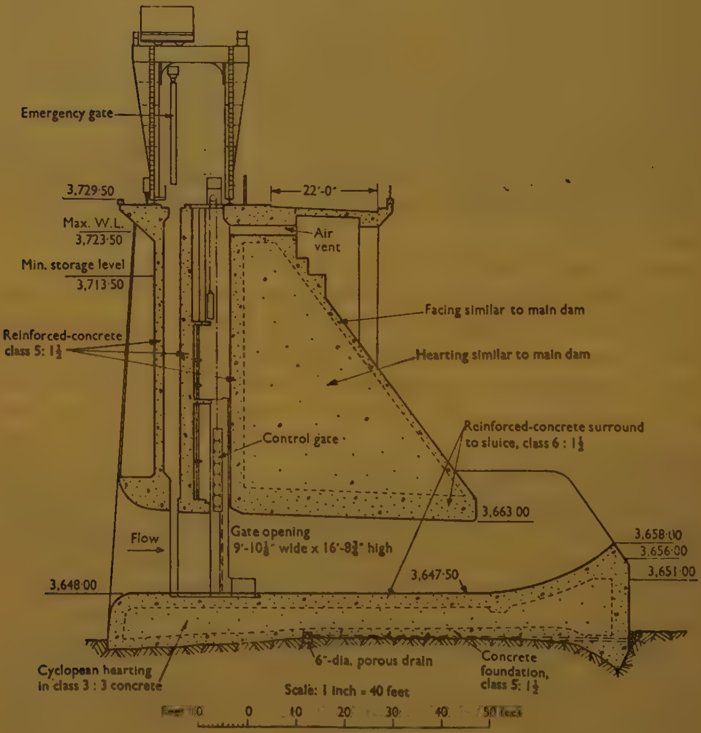
Investigation at an early stage had shown that the whole of the available water flow at Owen Falls could be utilized economically and in consequence turbine plant having a capacity to absorb it should be installed. On the other hand, facilities at the port of Mombasa and between there and Jinja imposed a limitation in the physical size of unit and, at the time the Owen Falls station was designed, the maximum rating of each unit was fixed at 15 megawatts; since then, however, the Railway Authority has put plans in hand which may permit the use of somewhat larger sets in future power stations.

The limitation to the size of the units, coupled with the desirability of absorbing all the power of the river, gave rise to a power station of considerable length for accommodation of ten 15-megawatt sets in the ultimate development. In order to minimize the cost, consideration was given to the possibility of designing the generating plant for outdoor service. It was estimated that although there might be some savings in the capital expenditure on the power station superstructure, these savings would be offset to some extent by the extra capital and operating cost (including maintenance) of the generators designed for outdoor use (*Fig. 8*).

Since the water intakes from the lake to the turbines were relatively short the power station could be combined with the dam to form an integral part of the structure and thus the problems of difficult regulation and hydraulic stability sometimes associated with long intakes at low heads did not arise. In consequence, no technical difficulties presented themselves in the choice of suitable types of turbines and associated plant.

Under the hydraulic conditions obtaining at Owen Falls, with a range in available head of 58 to 72 feet, both vertical-shaft Francis and Kaplan turbines were suitable for use with conventional alternators. The Kaplan turbine, however, is an adjustable blade propeller-type turbine, the main advantage of which lies in the relatively flat efficiency curve over a comparatively wide range of load and head. This feature makes it most suitable for conditions where variable head, discharge, and load have to be dealt with and, although somewhat more complicated and expensive than the Francis-type, it is possible to utilize a higher speed—in this case 150 revolutions per minute. The reduced cost and weight of the associated alternators, the space required in the power station building, and the crane capacity were among the many factors taken into account and these, coupled with the higher efficiency of the Kaplan turbine gave, in this case, a more economic installation.

Fig. 5


Emergency gate
3,729·50
22'-0"
Max. W.L.
3,723·50
Air vent
Min. storage level
3,713·50
Facing similar to main dam
Reinforced-concrete class 5: 1½
Hearting similar to main dam
Reinforced-concrete surround to sluice, class 6 : 1½
Control gate
3,663 00
Gate opening 9'-10⅛" wide x 16'-8¾" high
Flow
3,658·00
3,656·00
3,651·00
3,647·50
3,648·00
6"-dia. porous drain
Concrete foundation, class 5: 1½
Cyclopean hearting in class 3 : 3 concrete
Scale: 1 inch = 40 feet
Feet 10 0 10 20 30 40 50 feet

Section of Ground Sluice

Construction of Civil Engineering Works

The contract for the construction of the civil engineering works was awarded to a group of eight British, Danish, and Dutch firms in September 1949 as a result of International Tender. The contract was on a unit-rate basis with adjustments for variation in rates of labour, materials, and plant, whilst the contract time for construction was 4 years. The group of firms formed themselves into a company called The Owen Falls Construction Company. Details of this consortium are given in Appendix I of the companion Paper.[1] They had previously worked together on the reconstruction of Rotterdam Harbour after World War 2.

The contractors were faced with two main problems—first, the organization of large construction work on a far bigger scale than anything that had been attempted before in East Africa ; secondly, the diversion of the very considerable flow of the Nile during the construction of the main dam.

The contractors' first task was to increase the accommodation provided in advance for the labour by building a pilot camp for staff and artisans, married quarters for a certain proportion of the Africans, as well as 76 bungalows for artisans. The peak labour force reached a total of 200 Europeans, 123 Asians and 2,500 Africans. The requirements of the local health authorities entailed special precautions to reduce the risk of tropical diseases including the full-time service of a doctor, and the results fully justified the arrangements made.

The problem of programming and maintaining supplies of imported goods was considerable in view of the long rail haul of 750 miles from the nearest port of Mombasa, the congestion of traffic in the port, and the time of 3–4 months for delivery from the United Kingdom. Although over half the cement was imported, early in 1953 local supplies were obtained from a new works at Tororo, about 90 miles to the east of Jinja. Amphibolite rock from the excavations was crushed for the coarse aggregate, not an ideal material owing to the flaky nature of the product, whilst the sand consisted of one-third crushed amphibolite and two-thirds lake shore sand, transported 15 miles to a pier in Jinja by barges and thence by road to the work. The total quantity of concrete used was about 230,000 cubic yards.

The most difficult problem facing the contractor was the diversion and control of the river for the construction of the main dam.

Various alternatives were considered, such as diversion tunnels through the east bank, but it was eventually decided to adopt the simpler method of constructing cofferdams in stages across the river and diverting the flow first to one side and subsequently through temporary openings in the permanent dam.

Detailed information about the rock in the river-bed was largely unknown. Soundings showed that the rock ridge forming the falls on the curved alignment of the dam had a maximum depth of 25 feet under average river flow and fell away steeply upstream with a more gentle slope downstream. No borings were made in the main river channel, but those taken on each bank showed the amphibolite rock to be sound at a reasonable depth (see Figs 4, Plate 1). The assumption was therefore made that the forces of nature would have revealed any serious weaknesses in the rock ridge which should provide a safe foundation for the dam with the minimum of preparatory work. This was proved substantially correct when the cofferdams were dewatered. Some deep scoring of the rock parallel to the stream flow and an unexpected deep pocket of gravel proved troublesome but the difficulties were successfully overcome as described in detail in the Paper on constructional problems.[1]

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[1] D. P. Bertlin and Henry Olivier, "Owen Falls: Constructional Problems." See p. 670.

The stability of the cofferdams exercised the minds of the contractors, particularly if the river flow should approach the maximum of 43,000 cusecs. Reference has been made previously in the Paper to the natural barrier at the outlet of Lake Victoria, known as Ripon Falls, about 2 miles upstream. It was found that at comparatively low cost the river flow could be temporarily regulated at these Falls provided there was sufficient margin in the lake levels. A temporary sluice dam was, therefore, constructed across a rock spur on the right bank while the flow in the left channel was restricted with a rock barrier. This regulation prevented the flow at Owen Falls from exceeding 26,000 cusecs and, therefore, enabled the cofferdams to be considerably reduced in size and cost. Consideration was also given to the possibility of reducing the flow still further by a more substantial barrier at Ripon Falls. It was, however, a condition of an agreement with Egypt that a flow of not less than 21,000 cusecs should be maintained provided the lake level would produce such a flow without restriction.

It is interesting to note that during the construction of the dam, the lake reached a level 6 inches below the maximum recorded, corresponding to a flow of about 40,000 cusecs, but only a negligible proportion of the increase in level was caused solely by the Ripon Falls barrier.

The main cofferdam in the bed of the Nile was constructed in three parts starting from the left bank. First, a murram cofferdam was formed to a small island about one-fifth of the way across the river. This enabled an early start to be made on the excavation, and the river-bed was dewatered in this section by December 1950. The second-stage work comprised the construction of a crib cofferdam about halfway across the remaining channel to enclose part of the site of the permanent ground sluices, while a third stage covered the remainder of the work after diversion of the river through temporary openings in the completed foundations of the main dam.

Details of the methods of constructing and placing the cribs are given elsewhere.[1] Only a brief account of the main features is, therefore, necessary at this stage. The cribs were formed of timber logs bolted together, made in three separate sections for ease of placing. The bottom section was tailor-made to fit closely to the configuration of the rock ridge. These cribs were placed at 32-foot centres with steel beams spanning from crib to crib supporting a line of steel sheet-piles on the face.

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[1] See footnote 1, p. 637.

The bottom of the steel sheet-piles rested on the hard amphibolite rock and was sealed by a murram blanket extending upstream and itself protected from scour by rock rubble, and for a short length by a wall of sheet piles. Fissures in the rock and a deep gravel pocket were sealed with clay and concrete bags and were later grouted.

When placing the concrete in the main dam, temporary openings, 15 feet wide, were left at 30-foot centres to pass the flow of the Nile while work within the third-stage cofferdam was proceeding. The timber cribs were left in position on completion of the work except for those opposite the ground sluices.

The remainder of the dam was constructed in blocks 40 feet wide with gaps of 7 feet which were subsequently filled after a time lag of at least 6 weeks.

The construction of the rest of the work presented no special difficulties. Although the power-house foundations required a cofferdam only on the downstream side, the presence of a deep pocket of gravel at the end of the tail-race made it necessary to resite the power house about 90 feet upstream. This relocation effected a substantial saving in time and cost in constructing the murram cofferdam.

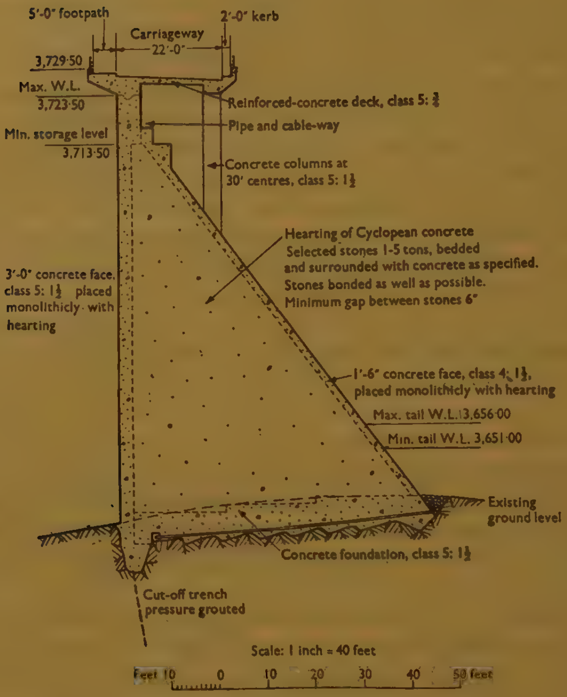
The course of the river automatically placed the buildings on an east–west axis which is the ideal orientation for tropical buildings. All fenestration thus occurs in the north and south walls and simplifies the provision of sun-protecting devices. Projecting heads and transomes to the power-house windows give adequate protection to this building. Additional protection from low-angle sun is provided in the office building by exposing the columns and extending the floor slabs beyond the face of the external walls (Figs 9, Plate 2).

The power station (see Figs 7 and 8, Plate 2) is a steel-framed structure with columns cased in concrete, and external walls formed in precast concrete units. The external face of the precast units on the downstream side is formed of a local pink granite aggregate $\frac{3}{4}$ inch maximum size which has been exposed by brushing away the surface of the slabs while they were still green. The slabs are generally 26 feet long, 4 feet high by 12 inches thick, those between the windows being moulded in a series of vertical flutes.

A particularly interesting feature of the precasting work on this building is the window unit which is 26 feet long and 10 feet high with deep projecting head, transom and sill, and two mullions.

Over the loading-bay doors a coloured faience roundel is recessed into the wall face and depicts the golden-crested crane, the emblem of the Uganda Electricity Board.

Fig. 6


5'-0" footpath
2'-0" kerb
Carriageway
22'-0"
3,729·50
Max. W.L.
3,723·50
Min. storage level
3,713·50
Reinforced-concrete deck, class 5: ¾
Pipe and cable-way
Concrete columns at 30' centres, class 5: 1½
Hearting of Cyclopean concrete Selected stones 1-5 tons, bedded and surrounded with concrete as specified. Stones bonded as well as possible. Minimum gap between stones 6"
3'-0" concrete face, class 5: 1½ placed monolithicly with hearting
1'-6" concrete face, class 4: 1½, placed monolithicly with hearting
Max. tail W.L. 3,656·00
Min. tail W.L. 3,651·00
Existing ground level
Concrete foundation, class 5: 1½
Cut-off trench pressure grouted
Scale: 1 inch = 40 feet
Feet 10 0 10 20 30 40 50 feet

SECTION OF MAIN DAM

ELECTRICAL GENERATION, SWITCHING, AND TRANSMISSION

The water-cooled, closed-circuit ventilated, vertical shaft alternators are designed to generate at 11 kilovolts which is stepped up through water-cooled transformers to a higher bus-bar voltage. A feature of these machines is that the thrust-bearing is located below the rotor, as the result of which some saving in the height of the building was obtained.

When the generating plant was ordered in 1948, no definite scheme could be developed for the transmission system, but at that time it was thought that the larger part of the output would be absorbed within a comparatively small area in the vicinity of the station and therefore the main part of the distribution would be over a 33-kilovolt network.

It was visualized, however, that later development of the system would be accompanied by the necessity to transmit energy over long distances, which would involve the use of a pressure of 132 kilovolts or even 220 kilovolts and the alternator characteristics were determined with this ultimate requirement in mind, including provision for synchronous condenser operation.

This requirement also made necessary the provision of excitation control equipment with a high rate of response and which would maintain stability under fault conditions. To this end an electronic voltage-regulator was developed which, on works tests, has shown its ability to hold the voltage remarkably steady under exacting conditions.

The necessity for high-voltage transmission for a large block of power has since materialized and consequently only the first four alternators are now being connected to a bus-bar operating at 33 kilovolts, whilst the remaining six machines will be connected through 11-kilovolt switchgear and step-up transformers to a bus-bar voltage of 132 kilovolts.

The auxiliary equipment associated with each turbine and alternator is operated from a unit transformer directly connected across the alternator terminals whilst the non-essential auxiliaries are supplied from step-down transformers connected to the main 33-kilovolt bus-bar. This supply also provides a standby for each of the turbine and alternator auxiliaries. A station battery is installed for control circuits and other services and is also arranged to give a supply to the lubricating-oil pump for starting and emergency use, whilst a separate turbine-driven governor oil pump is also provided.

The control indication and alarm system associated with each turbine, alternator, and step-up transformer is brought to gauge boards which are located in the machine hall on a gallery close to each alternator unit. The apparatus is equipped so that, in the event of a fault occurring in the associated turbine, alternator or transformer, the incident is immediately signalled on an illuminated section of the appropriate gauge board and the attention of the operator is drawn by means of an audible and visual alarm. Simultaneously a warning of mechanical or electrical trouble is carried to the control room.

The control room for the whole system, that is to say, generation as well as transmission, is situated in the office block (Figs 9, Plate 2) close to the machine hall. In the control room are placed the control panels for the 33- and 132-kilovolt switchgear and each alternator is provided with a separate panel which accommodates the exciter and turbine-speed control facilities. The engineer's desk will accommodate the alarm system to which reference has already been made, together with the illuminated sections associated with the switching station and the telephone and communication equipment; the latter includes a radio-communication system. Provision is also made in the control room for indication and recording of the head-race and tail-race water levels, sluice gate openings and summated turbine discharge.

An area on the west bank of the river adjacent to the power station has been levelled to form a site for the outdoor switching station which now accommodates the 33-kilovolt switchgear and which will ultimately receive the 132-kilovolt equipment also. Power is transmitted from here to Jinja and Kampala at 66 kilovolts whilst other lines of 33 kilovolts reach to Entebbe to the south-west and to Tororo and Mbale in the east, and to an industrial area which is being developed on the west bank of the river close to the Owen Falls site.

A further transmission system of 132 kilovolts has now passed the planning stage and one of these lines will be routed through Tororo towards the Kenya border and will thus be able to supply a considerable demand from that direction.

The alternators themselves are protected from fire by a carbon-dioxide installation which is automatically operated should an internal fault in the windings develop, and simultaneously the machine is shut down. External to the alternators, the large quantity of lubricating oil contained in each set, totalling nearly 3,000 gallons, which is used for the alternator thrust bearings and in connexion with the hydraulic governor and Kaplan-blade operating gear, is also protected by a large carbon-dioxide installation automatically operated by thermal releases. Whilst in both these cases the risk is considered to be extremely small, yet the results of a fire would be so disastrous that full protection was considered essential. In addition to these special provisions, a fire-water main runs down the length of the station and to other strategic points to which hydrants are connected and the system is so designed that the opening of any one of these hydrants automatically brings into operation the high pressure engine-driven fire pump located in the fire station.

ECONOMICS OF THE SCHEME

When, in 1947, the then Financial Secretary of Uganda moved the adoption of the Westlake Report in the Uganda Legislature, he referred to the Owen Falls Scheme as an act of faith. Provision was to be made at great cost for a hydro-electric station vastly in excess of any established demand. Many millions of pounds would have to be spent on the creation of new industries before the power available could be utilized.

Since the date of the decision to proceed, the costs of all essential materials and labour have risen greatly. It may now be said that the cost is approximately twice the original estimates. The cost of transmission also has proved to be much greater than was expected.

The Board have followed the usual policy in capitalizing interest during construction and this has greatly increased the total capital investment, but, as Tables 1 and 2 show, it is confidently expected that the revenue will reach, not later than 1962, an amount necessary to cover all expenditure including the whole of the interest on capital.

An important feature in the cost of colonial development is the provision of houses for most members of the staff. Approximately £1 million sterling has been spent on this one item alone.

Good town planning is to be admired, but it is not always apparent that the town planner has in mind the cost of public services. Much of the town planning in Uganda—from a town planner's point of view—is wholly excellent, but has involved extensive frontages to most premises. In a new housing development area in Kampala of about 1½ square mile, the Board has 602 consumers. The cost per consumer for the high and low voltage distribution averages £75.

The provision of adequate plant to cover the period between 1948 and 1954, when the hydro-electric station would come into commission, involved the Board in a decision of considerable importance. Diesel-engine-driven generating plant of 15,000 kilowatts, costing approximately £1¼ million was ordered. This bold decision has been fully justified. The availability of this thermal plant enabled extensive development of demand and the revenue during this period has risen from approximately £70,000 a year to an estimate for 1954 of £700,000.

The Board obtained their first loans from the Uganda Government; later these loans were supplemented by £2 million obtained on short-term loan from the London market. In 1950 the Uganda Government floated a loan on the Board's behalf for £3·1 million and in 1952 a further loan of £5,983,000 was raised. At the present time, considerable use is made of local capital resources, including a small amount of short-term borrowing. It is believed that these sources of finance will be adequate until 1955, when a further loan must be floated on the London market.

The Board has adopted the following periods for amortization of capital expenditure:

	Years
Acquisition expenses	60
Hydro-electric civil works	80
Hydro-electric plant	40
Thermal stations	20
Transmission lines	40
Distribution mains, services, and plant	30
Meters	20
Housing, offices, etc.	60
Furniture and fittings	10

Any hydro-electric scheme is faced with high annual capital charges. In the case of the Uganda Electricity Board, these charges have been considerably increased by reason of the works being carried out during a period of exceptionally high prices and also on account of the heavy burden of capitalized interest.

The cost of European staff is very high. A linesman costs approximately three times as much as a similar employee in the United Kingdom and a stenographer costs approximately twice as much. Not only must considerably higher salaries be offered to European staff, but in addition there are the annual charges in respect of medical expenses, housing, and the provision for leave-pay and free passages home and back again.

The cost of maintenance is, of course, substantially increased in that almost all electrical and mechanical material and equipment is imported and replacement of parts is more difficult and expensive than it would be if the manufacturers were close to the site.

As previously mentioned, the Board has experienced a very substantial increase in revenue since they commenced in 1948. The revenue for 1954, estimated at approximately £700,000, has been achieved before the commencement of supply to any of the major industries now envisaged and of the bulk supply to Kenya.

Tables 1 and 2 give a comprehensive picture of the Board's capital expenditure, revenue expenditure, revenue, units generated, and other interesting details. Table 1 gives the actual figures for the years 1948 to 1953, and Table 2 is the forecast for the years 1954 to 1962. The Authors are of the opinion that the estimates contained in those schedules are capable of realization, and in that event the average price per unit sold will drop from 2·8*d.* per unit in 1954 to 1·3*d.* per unit in 1962.

The following comparative statement shows the percentage of total revenue estimated to be contributed by each major class of consumer :—

	Year 1954 : per cent	Year 1962 : per cent
Industrial power	47	40
Bulk supply	—	30
Domestic premises	38	20
Hotels, clubs, etc.	8	5
Other users	7	5
	100	100

TABLE 1.—UGANDA ELECTRICITY BOARD STATISTICS, 1948 TO 1953

	1948 (7 months)	1949	1950	1951	1952	1953
Number of consumers .	3,263	4,143	5,298	6,836	8,783	10,400
Units sold : millions .	3·7	8·8	13·8	24·6	33·8	51·1
Maximum demand : kilowatts . . .	1,688	3,104	4,340	7,760	10,000	13,000
Revenue income : thousands of £ . .	43·0	93·4	152·6	272·7	368·5	555·6
Revenue expenditure : thousands of £ . .	43·7	98·5	142·6	229·0	346·6	492·2
Trading profit : thousands of £ . .	—	—	10·0	43·7	21·9	63·4
Trading loss : thousands of £ . .	0·7	5·1	—	—	—	—
Sale of energy (average per unit sold) : pence	2·65	2·41	2·56	2·58	2·56	2·58
Cost per unit sold (excluding capital charges) : pence .	2·83	2·68	2·47	2·23	2·46	2·31
Total capital expenditure : £ millions .	0·7	1·7	3·6	6·1	10·4	14·7

It will be seen from the above that the way in which the load is estimated to develop tends towards greater stability in revenue.

The tariffs at present in use in Uganda are based upon thermal generation. This decision has, of course, resulted in a tariff which has a relatively low fixed charge and a high unit charge. Clearly this type of tariff is not suitable for an undertaking in which the whole of the output in future will be derived from hydro-electric resources with its much greater capital charges and almost negligible running costs.

The period of transition from one type of tariff to another will be spread over a number of years. The high-load-factor consumer will prefer a higher kilowatt charge associated with a low unit charge, whereas the low-load-factor consumer, or the seasonal user, may be content to remain on a tariff similarly constructed to those at present in force.

The cost of fuel-oil in Uganda is high, resulting in the average cost per unit sold, excluding capital charges, of approximately 2*d.* It is thought that most domestic users will welcome a tariff having a high fixed charge with a low unit charge since the amount of electricity consumed in Uganda homes is considerably in excess of a comparable house in Great Britain. African household staff have not yet learned to be economical in the use of electricity.

The supply of electricity to Africans in rural areas presents a major problem. Since there are few integrated African communities in the Protectorate, it is not possible at present for economic reasons to supply African houses except in those places where African housing schemes have been built.

For bulk supply and metallurgical purposes the Board propose that supplies should be offered on a kilowatt, or kilovolt-ampere year basis with no unit charge. Most of the large-scale industrial development contemplated in Uganda will require power for electric-arc furnaces or electrode boilers. The cost to such users must be as low as possible since charges for electricity form a high proportion of their manufacturing costs. The extremely high load-factor of arc furnaces makes it possible to offer attractive tariffs.

When the Board started in 1948, the simultaneous maximum demand was only 1½ megawatt and it is estimated that by 1962 this figure will have increased to over 90 megawatts. In a rapidly developing area such as Uganda, the formulation of tariffs presents novel problems. The balance between differing types of consumers can only be a matter of conjecture. The pattern of possible demands is just beginning to emerge. An overriding factor in the fixing of rates is that consumers are asked to pay what the service is worth to them. There is nothing new in this.

Another interesting feature of the Uganda load, which has an important bearing on tariffs, is the high-annual load-factor achieved. This is due to a number of causes; the temperature range is small, there is no space heating demand, and darkness falls at approximately 7 p.m. throughout

TABLE 2.—UGANDA ELECTRICITY BOARD

Line No.		1954	1955
	GENERATION		
1.	Hydro-electric plant installed: kW.	60,000	90,000
2.	Maximum demand: kW.	15,200	17,750
3.	Units generated: thousands	69,600	76,400
4.	Load-factor: per cent	52·3	49·1
5.	Units sold: thousands	60,000	66,400
6.	Units sold: per £ of distribution capital	32	32
	CAPITAL EXPENDITURE (at end of each year)		
7.	Acquisition expenses	175	175
8.	Hydro-electric station	11,370	12,285
9.	Thermal stations	1,264	1,264
10.	Transmission	1,724	1,979
11.	Distribution	1,884	2,052
12.	Housing, offices, transport, etc.	1,448	1,632
13.	Loan expenses	1,059	1,059
14.	Total capital expenditure	18,924	20,446
	OPERATING AND MANAGEMENT COSTS		
15.	Generation	110	85
16.	Transmission and distribution	70	80
17.	Management	210	215
18.	Gross annual costs	390	380
19.	Deduct charges to capital	135	120
20.	Net annual costs	255	260
	SUMMARY OF COSTS AND REVENUE		
21.	Gross capital charges	769	862
22.	Net operating charges	255	260
23.	Total annual costs	1,024	1,122
24.	Revenue	700	774
25.	Balance charged to capital	324	348
26.	Surplus	—	—
27.	Gross capital charges per unit sold	3·076	3·115
28.	Charged to capital per unit sold	1·296	1·258
29.	Net capital charges per unit sold	1·780	1·857
30.	Operating costs per unit sold	1·020	0·940
31.	Total costs per unit sold	2·800	2·797
32.	Average price per unit sold	2·800	2·797

Note: Item 15 includes £30,000 for fuel in 1954

STATISTICAL FORECAST 1954 TO 1962

1956	1957	1958	1959	1960	1961	1962
90,000	90,000	90,000	90,000	105,000	120,000	120,000
27,300	49,100	60,900	69,100	81,200	87,300	94,000
105,700	193,100	242,000	279,200	334,800	369,300	405,500
44·2	44·9	45·4	46·1	47·1	48·3	49·2
92,700	172,400	216,000	249,300	298,900	329,700	362,000
40	69	81	88	99	103	108
Thousands of £						
175	175	175	175	175	175	175
12,918	13,180	13,367	14,417	15,013	15,136	15,136
1,264	1,264	1,264	1,264	1,264	1,264	1,264
2,092	2,127	2,145	2,145	2,145	2,145	2,145
2,305	2,490	2,665	2,840	3,015	3,190	3,365
1,745	1,871	1,887	1,902	1,917	1,932	1,947
1,059	1,304	1,304	1,304	1,304	1,444	1,444
21,558	22,411	22,807	24,047	24,833	25,286	25,476
95	105	115	120	125	135	150
90	105	115	130	140	160	190
220	225	230	235	240	250	260
405	435	460	485	505	545	600
110	35	25	25	25	25	25
295	400	435	460	480	520	575
911	1,084	1,124	1,164	1,225	1,307	1,350
295	400	435	460	480	520	575
1,206	1,484	1,559	1,624	1,705	1,827	1,925
873	1,175	1,354	1,474	1,639	1,779	1,935
333	309	205	150	66	48	—
—	—	—	—	—	—	10
Pence						
2·358	1·509	1·249	1·121	0·984	0·951	0·895
0·863	0·430	0·228	0·144	0·053	0·035	—
1·495	1·079	1·021	0·977	0·931	0·916	0·895
0·763	0·557	0·483	0·443	0·385	0·379	0·380
2·258	1·636	1·504	1·420	1·316	1·295	1·275
2·258	1·636	1·504	1·420	1·316	1·295	1·283

the whole year, with street lighting in use for approximately 12 hours a day. Water heating and refrigeration contribute substantially to the load and in turn have a high-load factor.

The load factors which will be achieved when the major industries are in operation can only at this stage be a matter of conjecture. Whilst the board will encourage short-term contracts for off-peak demand, they must always have regard to the maximum output in units which can be obtained from the Owen Falls Station, namely, 700,000,000 kilowatt-hours on the agreed maximum flow in the river, corresponding to a 60 per cent load factor on the fully developed station.

The Uganda Electricity Board have approved of the presentation of this Paper. The views and estimates expressed therein are, however, the personal responsibility of the Authors and are not necessarily those of the Board.

The Paper is accompanied by 6 photographs and 9 sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared, and by the following two Appendices.

APPENDIX I

MODEL TESTS ON GROUND SLUICES IN MAIN DAM

Model tests were carried out under the direction of Professor C. M. White, at the Imperial College of Science and Technology, London, to investigate the performance and behaviour of ground sluices located in the main dam.

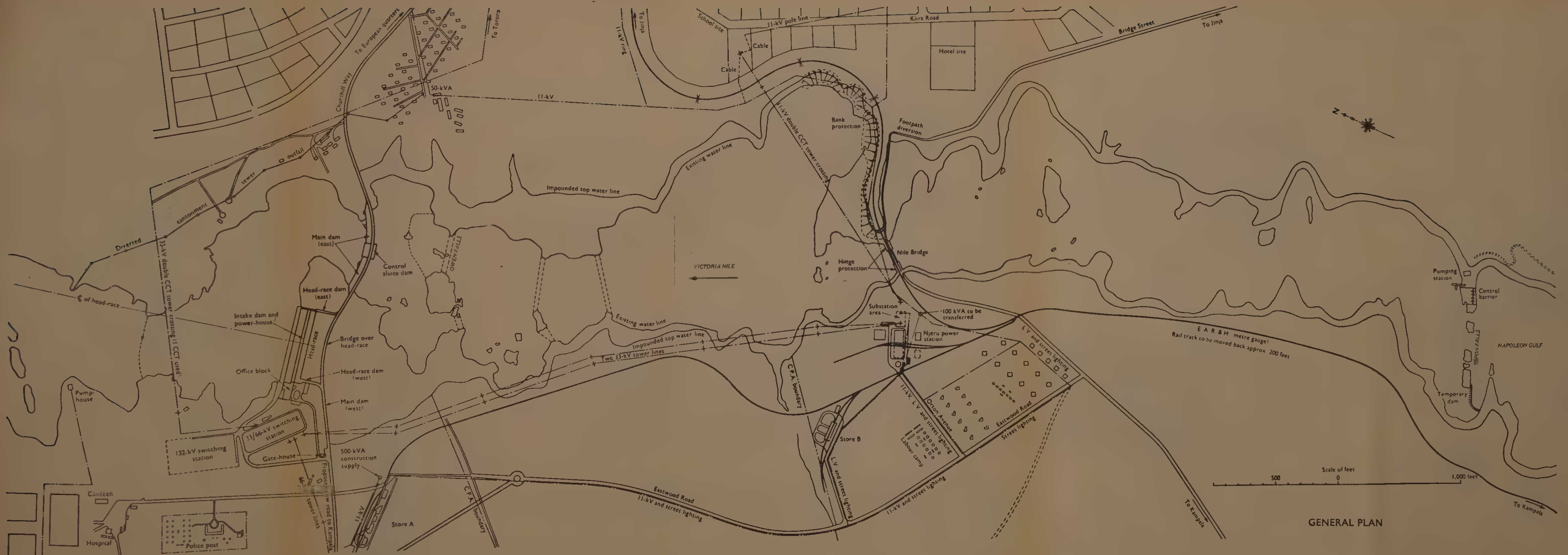
Near the centre of the Owen Falls dam, sluice gates are arranged side by side in six tunnels separated by walls 2 metres thick. Each gate is 5·1 metres high by 3·0 metres wide, and has an area of 165 square feet (988 square feet total for six). However, the minimum cross-sections of the six sluiceways add up to a total of 915·5 square feet, and the calculation of coefficients is based on the latter. The gate sills are at elevation 3,648·0 feet, and the upstream level ranges from 3,711·25 to 3,723·5 feet. Correspondingly the capacity of the sluices ranges from 41,000 cusecs to 46,000 cusecs (1,160 cumecs to 1,310 cumecs). Concentrated on a width of little more than 100 feet, the six sluices together may be called upon to discharge substantially the whole flow of the river and may do so for long periods of time, and at a speed of 60 feet a second. Compared with this, the greatest speed in the natural falls is perhaps 20 feet a second and the width is nearly 400 feet. The greater concentration and the greater speeds from the sluice must inevitably increase the risk of attack on the river bed, and the sluiceways are provided with up-turned ends to keep the main point of attack well away from the dam. Accordingly experiments were undertaken to ascertain the place of attack and to find a good compromise concerning the level of the tunnels and the angle of up-turn.

The physical thrust of the discharge from the sluice is great. At 65 feet a second the force of the jets amounts to more than 2,500 tons, concentrated within a width of 100 feet. Ultimately this force is dispersed and transferred to many thousands of square feet or river bed. A considerable experimental programme was undertaken to find how to transfer the force smoothly to the bed without setting up violent currents or great waves in the process.

Many further problems concerning the diversion of the river during construction of the dam were also investigated in detail, including the placing and mooring of cribs

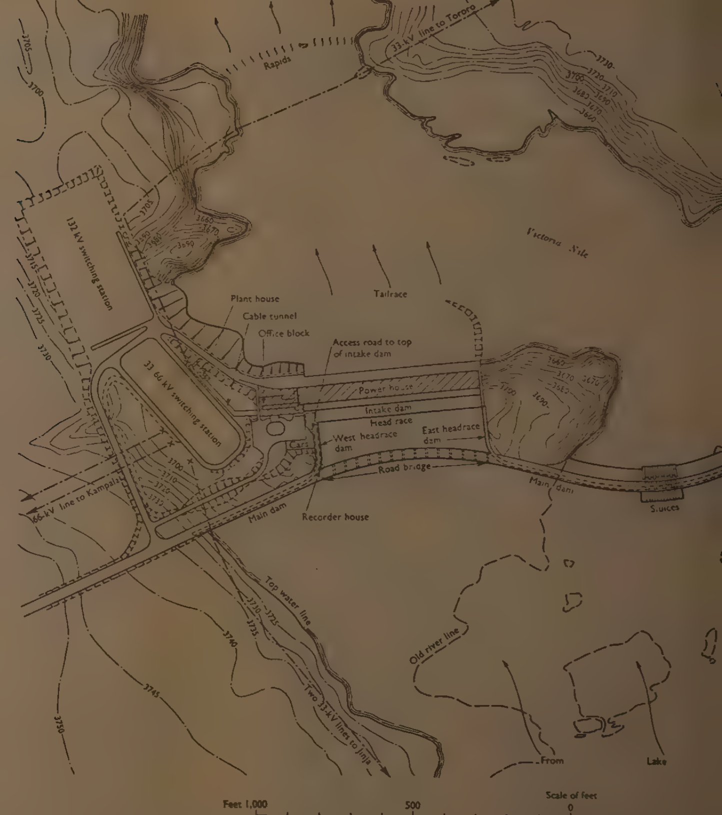
OWEN FALLS, UGANDA, HYDRO-ELECTRIC DEVELOPMENT

FIG. 2



GENERAL PLAN

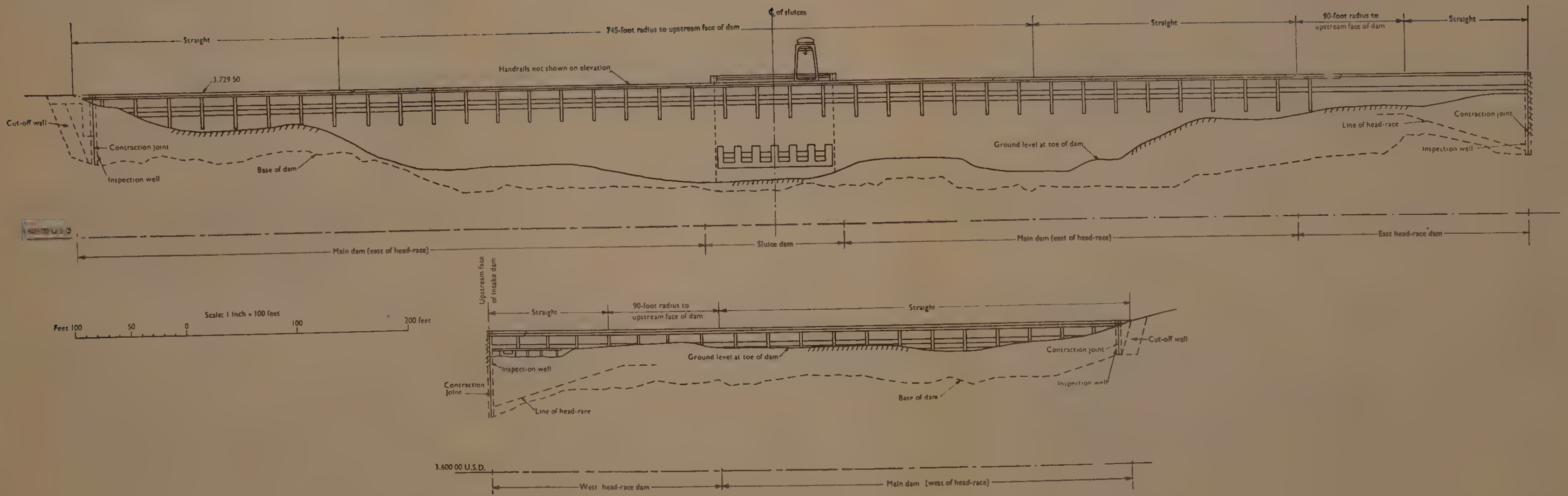
FIG. 3



SITE PLAN

The Institution of Civil Engineers. Proceedings, Part I, November 1954

FIGS 4



DEVELOPED ELEVATION OF DOWNSTREAM FACES OF MAIN DAM AND SLUICE DAM in fast water. Alternative methods of construction and their reaction upon the diversion works were explored.

The investigations outlined above were carried out in the Hawksley Hydraulic Laboratory of Imperial College between April 1949 and May 1950. Three models were built of scales of 1 : 30, 1 : 60, and 1 : 100 respectively. The first two were undertaken on behalf of Messrs Sir Alexander Gibb & Partners and the third on behalf of The Owen Falls Construction Company.

In view of the variety of organizations interested in the discharge of the sluices, a model of a single sluice and tunnel together with upturned exit was built mainly of transparent plastic to the scale of 1 : 30. At this scale the gate in the model was approximately 7 inches by 4 inches, large enough to accommodate measuring devices for studying the retardation caused by the walls, an effect required for predicting scale corrections. Volumetric measuring-tanks enabled the discharge to be ascertained within one part per thousand, so eliminating one of the many uncertainties inevitable in experimental hydraulics. This model confirmed the action of the upturned toe, which, while throwing the jet a good 90 feet from the structure, did so without causing backwater and the associated hammering of the gate except when it was less than 2 feet open. The gates normally will be opened fully. When fully open the discharge coefficient of the single gate, uncorrected for scale, was 0·828, the head being reckoned from a datum of 3,664·7 feet which is 16·7 feet above the sill, and the area being that of the greatest constriction, namely 152·6 square feet. For six gates discharging together the coefficient would be 0·820 and each gate would contribute 7,700 cusecs when pond level was 3,723·5 feet, or the total discharge would be 100 million cubic metres a day at lowest pond level of 3,711·25 feet. The tunnel was "pressure plotted" but no undue risk of cavitation was detected. The pressure plotting disclosed the amount by which the upturned toe reduced the discharge and showed dissipation to be only a minor action. Guided by this, several modifications of the tunnel were investigated, some of which had appreciably greater capacity.

The main work was done with a more comprehensive model including nearly 900 feet of the dam, 250 feet of the upstream pool, and 750 feet of the river downstream. This was built to scale 1 : 60, and the model dam was then 20 inches high and about 15 feet long. The dam was made of concrete, cast in 9-inch segments which, if necessary, could be moved to different positions, and the river bed was moulded to templets in a weak mix of coarse sand and cement to represent inerodible rock. Inadvertently, a dry rubble filling was used below part of the bed, and subsequently gave much trouble. The gate-house section of the dam was cast in six vertical sections with joints in the plane of the eastern side of the tunnel, a particularly good and simple method of construction attributable to Mr J. R. D. Francis of the Hawksley Laboratory. Restricted in length by the shortness of the only test basin available, the model was equipped with diffuser and baffles in the upstream pond, and with an adjustable weir oriented diagonally at the downstream end, arranged to draw the water as in nature. Separated by no more than 2 metres, the sluices might be expected to interfere with one another's calibration. However, as is evident from Table 3, the effect is not great.

TABLE 3.—INTERFERENCE OF THE SLUICES UPON ONE ANOTHER

Number of sluices open	1 : 60 model. Change in calibration (TWL) compared with all six open : per cent
1, 2, and 3 together	−0·8
2, 3, and 4 together	−0·6
2 and 3 together	−1·2
1 and 2, or 5 and 6 together	−0·7
1 or 6 alone	+0·3
2, 3, 4, or 5 alone	+1·3±0·4*

* Standard deviation : 0·3 per cent.

Vortices were sometimes seen in the upstream pool. Most submerged intakes are liable to them, and the air drawn in may be very objectionable, for instance, when the intake supplies a turbine. However, in the present ground-sluices, this is unimportant unless the vortex is big. Small ones in the model formed occasionally. No doubt they will be there in nature whenever winds and temperatures are adverse. In the model they did not visibly change the upstream level; evidently they were too weak to reduce the discharge.

Downstream from the dam, the jets from the sluices set up violent currents which swept great boulders around, even at places 1,000 feet distant from the sluiceways. Waves 5 feet high occurred, superimposed upon slow surges sometimes 2 feet high with a period of perhaps 45 seconds. Currents of nearly 30 feet a second were observed along the west bank of the river, and of 7 feet a second along the dam. The jets swept the water away, and exposed the bed to the full strength of the jets. At Owen Falls, perhaps this may not be objectionable, since there is incontrovertible evidence that the rock is exceptionally resistant to hydraulic erosion. However, various alternative locations for the sluices were investigated, and pools of various dimensions were excavated in the bed where the jet impinged. In this way an arrangement was found, in which a substantial fraction of the thrust of the jets could be carried locally by the walls of a pool 10 feet deep. The pool reduced the waves and surges to less than 1 foot in the vicinity of the power-house. By various experiments the original location and orientation of the gate-house regarded as a unit was confirmed to be as effective as possible in reducing the attack on the banks of the river.

Other experiments on diversion works for the contractors, carried out in a more comprehensive model of smaller scale, were designed and supervised by Mr Bijl of the Construction Company, who, in the light of an unique opportunity for subsequently observing the action of the full-scale river during construction, may perhaps in due course describe this aspect on the investigation.

The great number of experiments described could not have been done in the short time available without the fullest co-operation of all concerned. The laboratory staff was greatly helped in the organization, design, construction, and operation of the models by many members of the staff of Sir Alexander Gibb & Partners and particular acknowledgements should be made of the services of Mr Curry, Mr David Kent and Mr Fells who carried the load of most of the experimental measurements.

APPENDIX II

MODEL TESTS ON TURBINE INTAKES

In order to determine the best form of the turbine intakes, model tests were made under the direction of Dr E. F. Gibbs at Bristol University.

The quantity of water available at the laboratory was limited to 0·5 cusec, and for a maximum turbine discharge of 3,620 cusecs, a scale of 1 to 36 was accordingly adopted for the model.

Examination of the initial design of the turbine intakes indicated that the following effects might require investigation :—

(1) Flow separation at the bottom upstream edge of the inlet.
(2) Deceleration of the flow at the entrance to the transition from a rectangular to a circular section at the spiral casing inlet.
(3) Effect on the velocity distribution of :—
    (*a*) The transition section.
    (*b*) The fairly sharp curvature of the intake in the vertical plane.
    (*c*) The central splitter wall supporting the two control and bulkhead gates, which might cast a shadow at the entrance to the spiral casing.
    (*d*) Sharp corners in the rectangular portions of the intake.

The design of the first model took these points into account so far as possible and it was considered unnecessary to incorporate the turbine spiral casing and draft tube for these tests.

DESCRIPTION OF FIRST MODEL

(1) The approach channel consisted of a steel tank, approximately 4 feet long by 2 feet high by 2 feet wide, with its floor inclined at 45° corresponding to the maximum anticipated slope of the head-race. The downstream wall of the tank was inclined, to represent the original sloping face of the intake dam, and the opening in this wall for the intake was placed off-centre so that asymmetrical entry conditions, corresponding to an adjacent machine being shut down, might be investigated. A removable baffle could also be fitted to give symmetrical flow at entry.

(2) The intake itself was made in xylonite. The single curvature sections of the transition from square to circular were made of sheet brass, and the double curvature sections were made in copper gauze which was then tinned over, covered with filler, and enamelled.

TEST RESULTS OF FIRST MODEL

Velocity measurements were made by Pitot tube in four directions mutually at 45° at the downstream end of the transition, and showed the existence of a region of low velocity downstream of the splitter wall, together with fluctuations of unexpected magnitude. Various steadying devices were tried upstream and downstream of the model without effect, and it was found that the flow at inlet was not following the contours of the roof and sides, the consequent backflow and formation of vortices causing rotation of the flow in the intake passage and periodic drawdown of the surface. It was therefore decided to lower the top of the intake and flare the entrance. With these modifications, reasonably steady and repeatable velocity readings were obtained, but negative pressures and rotation of the stream still occurred, although some improvement resulted from adjustment of the head-race floor.

DESCRIPTION OF SECOND MODEL

Experience with the first model led to changes in the intake design, including the following new features :—

1. The face of the dam was made vertical to improve entry conditions, particularly at low head-race levels.
2. The gates were re-located to provide a longer transition section.
3. The intake floor was lowered and straightened, and the gradient of the head-race was made the same as that of the intake floor.
4. A single control gate was used with the splitter wall supporting only the common grooves for the bulkhead gates and screens.

TEST RESULTS OF SECOND MODEL

Observations on the second model (constructed of " Perspex ") incorporating these improvements confirmed that :—

1. The tendency to drawdown at low levels with full discharge was much less pronounced.
2. The velocity readings were reasonably steady and repeatable and the distribution was acceptable and free from rotation.

The model was used to determine the most suitable location for the current meters and supporting frames, etc., for efficiency tests, and it was decided that this measuring section should be located in the square section of the intake immediately downstream of the control-gate grooves.

Discussion

Mr R. W. Mountain, introducing the Paper, said that he and Mr Paton were very sorry that owing to illness Sir Charles Westlake was unable to be present. Sir Charles had asked him to point out, as a matter of interest, that when the first hydro-electric set was commissioned the load had almost reached the installed capacity of the thermal plant; he had also asked him to say that since the Paper had been written the second alternator had been commissioned—only a few days before the opening of the station by H.M. The Queen on the 29th April. Sir Charles also wished to draw attention to the reference on p. 644 to a bulk supply to Kenya; since the Paper had been written the negotiations for that bulk supply had proceeded and it was hoped that agreements would be signed very shortly.

All three Authors would like to be associated with the statement that they desired to place on record their high appreciation of the work carried out by the contractors, which included getting the materials and equipment to the site to enable the dam to be built and the plant to be erected almost within the scheduled time.

Mr Paton and Mr Mountain would like to add a personal note; they wished to pay a tribute to the work of Sir Charles Westlake over the past 7 years, and they thought that the Paper would show that a very large part of the success in bringing the scheme to its present stage was attributable to the energy and leadership of Sir Charles.

Lord Reith, opening the discussion, said that if he had to sign documents in which it was necessary to indicate his profession the only designation he ever put was "Chartered Civil Engineer"; and he was very glad he could so describe himself.

Last year he had had the great pleasure of being taken over the Owen Falls job by Sir Charles Westlake. It was one of the most interesting and thrilling days he had spent; the only one of that sort which had interested and thrilled him more was at the Panama Canal.

Listening to the Authors introducing the Paper, he had felt himself in envy of them; and the same would have applied the next night with the second Paper. Those gentlemen were presenting to the Institution and their associates a record of work and achievement which nobody could vitiate—as happened with work and achievement of other sorts; the Owen Falls dam would stand as a monument to their initiative, ingenuity, perseverance, and courage.

Sir John Hall, present that evening, had done more than anyone else, as Governor of Uganda, to get the project undertaken; he was to be congratulated. So was Sir Charles Westlake, solely responsible for the direction of all aspects of the job. So were Mr Paton and Mr Mountain, entrusted with the design and supervision of execution. So was Mr Bertlin and the construction company which he had managed with such efficiency.

Lord Reith had an immense admiration for, and envy of, all who had been associated with the great work; he hoped they would move on to other works already visualized.

Mr T. G. N. Haldane observed that the most remarkable feature of the Owen Falls scheme was the enormous amount of storage. He used to think that there never was a hydro scheme where one would not be glad of a little more storage, but for the Owen Falls scheme there was more storage than could even be hoped for. He was fascinated by the Authors' remark to the effect that if by any chance there had been a miscalculation about the size of the spillway and there were floods which might eventually result in over-topping the dam, there would be plenty of time to enlarge the spillway before any disaster occurred. That, he thought, was a unique feature of the scheme.

On p. 632, some very interesting information was given. It would be noted that the flow down the Nile was a very small percentage of the total run-off in the catchment area. The Authors had pointed out that it was possible that the flow would be appreciably increased by drainage of certain swamps. If such drainage took place—and it was necessary to look forward 50 or 100 years when considering a scheme of the kind in question—the flow of the Nile might be very considerably augmented, and he wondered whether sufficient means had been provided for increasing the installed generator capacity in those circumstances. He noticed that the sets were of 15,000 kilowatts, and that that size had been decided on owing to certain transport difficulties. If the evaporation losses were even slightly decreased, the flow down the Nile might be very considerably augmented, and the station would already be running at a load factor of 60 per cent when the full 150,000 kilowatts was installed. Was there any provision whereby that installed capacity could be increased in the future, possibly by using larger sets if by that time the transport difficulties had been overcome?

At the foot of p. 632, Mr Haldane had noticed that compensation was being paid to the Egyptian Government because of the reduction in the flow while the storage was being built up in the lake. He was left slightly puzzled by what was stated there; could the Authors say what would happen when the storage had been built up in a number of years' time? By that time, he would have thought, the flow would revert to its present average of 22,100 cusecs, unless, of course, there was a big increase in the evaporation losses.

Table 2 gave an excellent bird's-eye view of the economics of the scheme. Possibly it might be felt at first sight, on looking at that Table, that the scheme could have been postponed for a few years until the load had been built up; but, as the Authors had pointed out, the whole scheme had been a great act of faith, and he did not doubt that that faith would be fully justified in the result. He would also comment on the fact that the increase in cost, to which attention was drawn in the Paper had been attributable at least in part, and possibly largely, to the rise in prices which occurred in the interval between preparation of the scheme and completion of the works. That rise in cost worked against the scheme during that period, but if, as was not unlikely, there was a continued rise in price levels and a fall in the value of money, the same factor would be working in favour of the scheme in the future and might have very big effects on the overall economics. During the past 50 years the value of money had decreased fourfold, and it was quite possible that something of that sort would continue into the future.

Mr J. K. Hunter's remarks were not directed to the Owen Falls scheme in particular but related to hydro-electric schemes everywhere. The Authors had referred, he said, to the effect of unproductive capital employed during the period of construction and the subsequent load build-up. Those effects were brought out in the figures given in the Paper, and represented one of the most serious disadvantages of water power in relation to thermal power in those conditions where the promoters were faced with the necessity of building up their load from scratch.

Broadly speaking, the cost of water power was reduced as the scale of development was increased, and it would commonly be found that the larger the scheme the cheaper would be the resultant power; that was to say, the actual cost of construction per kilowatt, other conditions being equal, would tend to fall as the size increased. Schemes which were situated on large rivers did not as a rule lend themselves to development in easy stages, and so the promoters were faced with the difficulty of matching capital expenditure with earning capacity during the early years.

He recalled that some 25 years previously one of the Indian Provincial Governments had embarked on the hydro-electric development of one of the Himalayan rivers. The scheme had been sanctioned on the basis of the estimates of cost and the estimates of revenue, and on paper it looked very attractive. The scheme proved to be no exception to common experience in such matters, and the estimates had been considerably exceeded; but what had been equally unfortunate was that the revenue failed to match up to the forecasts, and by 1935 the financial position had become serious. Shortly afterwards, however, the war had broken out, and within a very short time the picture had completely changed. The unused power proved to be a godsend to the war industries which, at that time, had sprung up all over Northern India, and the general inflation which had followed soon allowed the scheme to be put on a sound financial footing.

An example of a different kind was provided by the Galloway Water Power Company, which constructed a group of five power stations in south-west Scotland in the early 1930's. There the promoters had been fortunate in being able to enter into an advance agreement with the then Central Electricity Board to purchase, apart from certain local supplies, the entire output from the scheme as soon as it became available, on terms which would cover all the costs involved. The capital expenditure thus became fully productive as soon as the stations had been brought into service. The promoters had, perhaps, been equally fortunate in that the works had been carried out during a deflationary period, and in the event the final cost of the scheme had exceeded the original estimates by only 10 per cent, an achievement which for the type of work in question had not been matched anywhere else, Mr Hunter suggested, either before or since.

The additional expenditure had been approved by the Central Electricity Board and since the actual output of the scheme had proved to be about 24 per cent more than the estimates, everybody had been happy. Today the British Electricity Authority had fallen heir to that undertaking, created by the courage and foresight of the original promoters and their engineers. The present-day value of the power provided by that 20-year-old scheme was between four and five times its cost.

Fig. 16 illustrated how a protracted period of load development might increase the real cost of power. It related to a purely hypothetical scheme, constructed to full development over a period of 5 years. The completion of the scheme was assumed to be followed by a 10-year period of steady load-development, at the end of which the sale of power represented the full output.

Fig. 16 was based on an interest rate of 3 per cent, and showed that at the end of the construction period the interest on the capital employed had increased the direct cost of construction A–B by 9 per cent. It was assumed that as soon as the scheme was brought into service the overall annual cost could be represented by 5½ per cent of the total capital cost A–C, that percentage being made up of 3½ per cent to cover interest and amortization of capital, and 2 per cent to cover the running costs.

The total annual cost at any time could be expressed in the following form:

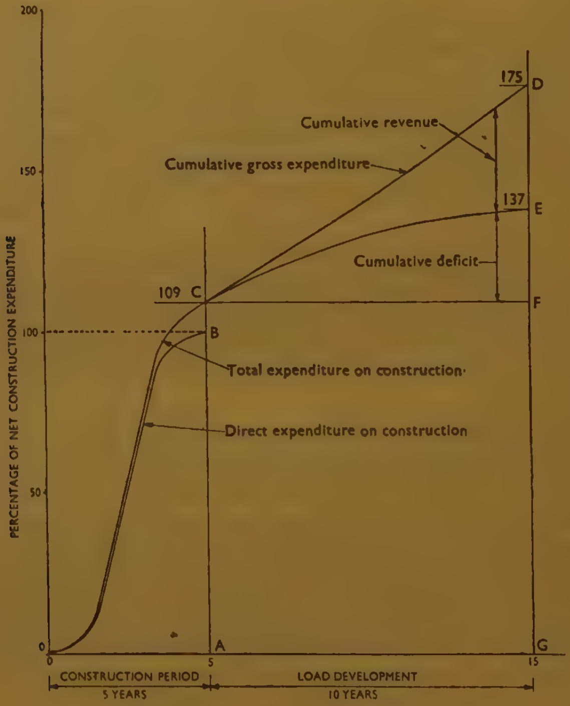
$$3\tfrac{1}{2}\%(Y + y) + 2\% Y$$

where Y denoted the total capital investment on completion of the works (AC) and y the subsequent increase in capital due to the accumulation of annual deficits (EF). If those annual costs were added to the capital AC the curve CD represented the accumulated expenditure. The accumulated earnings, which were assumed to increase at a uniform rate, could then be plotted downwards from CB to give the curve CE; the difference between the curve CE and the horizontal line CF represented at any time the accumulated deficit to date.

In the diagram, the average price at which power was sold at the station bus-bars had been selected to ensure that at the end of the 10-year period of load development a balance was achieved between revenue and overall annual cost, after taking into account the interest which had to be paid on the accumulated deficit; that was to say, the revenue earned during the tenth year of operation was equal to the total annual cost.

It would be seen that in the example chosen the relative capital employed at the end of the period of load development had increased from 109 to 137 or by 26 per cent.

Fig. 16



EFFECT OF UNPRODUCTIVE CAPITAL ON COST OF POWER

Total annual cost in terms of the actual cost of construction was, therefore :—

$$(3\tfrac{1}{2}\% \times 137) + (2\% \times 109) = 6{\cdot}98\%.$$

Had it been possible to find a market for the whole output of the scheme as soon as it was completed the total annual cost would have been $5\frac{1}{2}\% \times 109 = 6{\cdot}00\%$. From which it followed that the increase in the cost of power was $16\frac{1}{3}\%$.

That increase was a measure of the effect of the unproductive capital employed during the 10-year period of load development.

On a similar basis, but with interest rates at 5 per cent, the cost of power would have had to be increased by 28 per cent to cover the effects of a 10-year period of load development.

Had the period of load development been 20 years, the capital (at 3% interest) would have increased from 109 to 174, or by 60 per cent, corresponding to an increase in the cost of power of 38 per cent.

Had the interest rate been 5 per cent, a 20-year development period would have increased the cost of power by 88 per cent.

Professor A. H. Naylor said that credit for the Owen Falls scheme was due to Messrs Preece, Cardew and Rider and to Messrs Coode, Wilson, Mitchell and Vaughan-Lee. As far back as 1927, the late Mr Mitchell, of Coode's, in a report to the Governor of Uganda, had suggested a dam lower down than the Ripon Falls. It was not, however, until 1934 that those firms had been asked to report on hydro-electric power. With the late Mr A. E. Hughes as electrical engineer, Professor Naylor had been appointed civil engineer in charge of the investigation. The Governor of Uganda at that time, Sir Bernard Bourdillon, had impressed on them that Uganda was a protectorate, and that it was not the Government's policy to have the country industrialized because of the fear of causing discontent amongst the natives and taking labour away from essential areas; that there was not enough labour to cope with the forces of nature. That that fear had been justified was shown in the Paper, where it was stated that it was not economic to supply the rural areas. The only prospect of a large hydro-electric scheme had lain in the expected demand for power from the Kakamega goldfields just over the border in Kenya.

Accordingly, two minor and three major schemes had been prepared, the latter being Jinja Bridge, Owen Falls, and Murchison Falls. The decision as to which was best had rested on the maximum possible demand under the restrictive policy of the Government, but it had been shown that with a sufficiently large demand Owen Falls would be the most economic, and that, in contrast to the Murchison Falls scheme, it had the advantage of utilizing Lake Victoria as a reservoir. It had also been pointed out that there was a possibility of using the large area of Lake Victoria for regulating the flow in the interests of Egypt and the Sudan.[1]

However, the hoped-for demand for power did not materialize. Professor Naylor had had occasion to write to General Smuts, and in his reply, in sharp contrast to Sir Winston Churchill's sentiments, General Smuts had stated "I also rejoice (with you) that the primeval peace of Central Africa will not be disturbed just yet. Certain developments may well wait for a more distant future. Meanwhile, the mountains and valleys of Abyssinia are re-echoing with the monstrous noises of our civilization. One could have wished that there too silence had remained undisturbed. As it is, the din is spreading far beyond Abyssinia."

When, after the war, the Colonial Office had reversed its policy, it had been Messrs Preece, Cardew and Rider and Messrs Coode's who had nominated Mr C. R. Westlake as electrical engineer and Mr E. V. Richards as civil engineer, to report again in the light of changed conditions. Mr Richards's report of January 1947 had endorsed the Owen Falls recommendations for a major scheme, and Mr Westlake, reporting in May 1947, had accepted the suggestion of storage for Egypt, thus making it possible to plan on the basis of the whole mean flow.[1]

It was not clear to Professor Naylor why it should have been necessary to restrict the flow to 17,700 cusecs; he would have thought that it was in the interests of all concerned, and especially Egypt, to cut off the flow completely for a short period during construction and whenever possible thereafter, as, for instance, when the flow in Egypt was above normal, to limit it to the amount which would actually be required for power. In that way it would have been possible to build up the requisite volume of storage at the fastest rate so that full mean flow would have been available for all at the earliest possible date. Owing to the equalizing effects of Lake Kioga, Lake Albert, and the Sudd area, temporary cessation of flow in Uganda would hardly have been detectable in Egypt and the amount of any long-period diminution would have been greatly reduced or indeed completely masked if the flows lower down had been above the average.

In his recommendation he had advocated the siting of the Owen Falls dam at the lowest fall only if permission could be obtained for interruption of the flow of the Nile, so that the foundations could be placed in the dry. The Owen Falls proper—the next Falls up—had resisted deep scour, and therefore not only gave a higher rock bar but also the probability of a sounder rock bar. In addition, a large centre island would have facilitated construction. The disadvantage of the lower site with the need for high cofferdams and with deep gravel pockets was borne out in the event. In his opinion the choice of the upper site would have reduced the quantity of concrete and greatly reduced the cost. The increased length of turbine dam required could have been provided by prolonging it through the centre island towards the eastern bank. After the dam had been brought to its full height the necessary rock excavation in the tail-race could have been carried out intermittently in the dry.

On p. 634 it was stated that the dam had been "designed with a slight curve against the pressure of water," and it would be interesting to know whether with that large radius it had in fact been designed as an arch dam.

The almost complete separation of the power house, instead of its incorporation in the dam, seemed to be an expensive way of countering vortex tendencies.

Mr L. D. Anscombe said that although the choice of a large number of comparatively small units had been dictated by transport limitations, it should be borne in mind that the units in question were somewhat larger in physical size than their rating would normally indicate. The alternator characteristics had been wisely specified on a very generous scale, and the machines had thus been designed for low temperature rise, low reactance, and high inertia-constant, with the result that they were particularly suitable for the projected extensions in the transmission system. The machines were, in fact, of a frame size which might be capable of 25 megawatts in other circumstances.

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[1] Preece, Cardew and Rider, and Coode, Wilson, Mitchell and Vaughan-Lee, "Uganda Protectorate Hydro-Electric Investigation of the Victoria Nile." June 1935.

Nevertheless, he thought that it was true to say that even under such an arduous specification it would be easy to design machines of double the capacity without excessive physical dimensions. There would inevitably have been some saving in overall cost had that been possible.

The Authors had referred to some saving in the height of the building arising from the use of generator thrust-bearings beneath the rotors. That was a well known construction, but in the instance in question, because of the small diameter of the rotor in relation to the large thrust-bearing, which was designed to carry 465 tons total thrust, it had not been possible to drop the rotor rim and poles around the casing to form the umbrella arrangement which was usual in such cases. He felt that had larger sets been installed it would have been possible to adopt the true umbrella construction, with a further saving in height.

Another feature which had contributed to a reduction in height was, strangely enough, the quick-response excitation system used. That excitation system really arose, he thought, in the first instance from a desire to reduce the height of the station. The system was characterized by the fact that there was only one auxiliary machine on the shaft of the units in place of the three normally fitted, and that one machine, a compound-wound direct-current generator, supplied a separately-driven booster set connected in series to obtain variable excitation. The governor pendulum motor was fed from it through slip-rings, and by making the single generator of a flat pancake design, a considerable saving in height of the complete unit was achieved.

In planning a similar scheme in future, would the Authors be in favour of investigating the outdoor type of station ? It need not be of the completely outdoor type, in which the machines would stand outside much as they did within the station now, in casings above a floor level at the base of the stators, but might preferably be of the semi-outdoor type in which they were located in pits under light covers. There would be a maintenance pit and a gantry crane serving the whole station. He believed that considerable savings would result from that arrangement, and there would be little disadvantage in relation to servicing and weather conditions. Unfortunately, however, the appearance of such a station could hardly be expected to reach the standard which had been achieved at Owen Falls.

Professor C. M. White remarked upon the great size and potentialities of the catchment on which the Owen Falls project was based, extending as it did to 100,000 square miles, the greater part of which was subject to very generous rainfall. He thought the exact figure for the rainfall remained a mystery. In the Paper the Authors had suggested that the average was 43 inches. Yet at the proposed flow of 17,700 cusecs, the Owen Falls project was using only 2 inches of run-off, which was a remarkably small fraction of 43 inches of rain.

One explanation was no doubt the size of the lake. The lake was one-quarter of the catchment, and evaporation from the surface, the lake being on the equator, was presumably not less than 60 inches, and indeed might well be as much as 120 inches, which could account for the difference; but, in doing so it emphasized that a flow of more than 100,000 cusecs was entering the lake. Since a considerable part of the catchment whence it came was more than 2,000 feet above the lake, there were likely to be many places round the lake where hydro-power schemes of great magnitude could be built.

Turning to the spillway problem, he said that simple though spillways were, they were probably the most troublesome part of hydro-electric schemes. There were, he supposed, more different kinds of spillways and more variety in their design than of any other part, which was good evidence that none was really satisfactory! No spillway was really a dissipator. To dissipate the energy of the jet one would have to disperse it, reducing its velocity, and the structure would have to have the same cross-section as the river. The purpose of the ski-jump type of toe as used at Owen Falls, was to push the point of dissipation as far from the structure as possible. Certainly, even in rock, a hole would form where the jet came down, but the hole itself helped to quieten things. The material from the hole could sometimes present a problem, because it might build up a second dam, which at certain sites, could interfere with the outfall from the turbines.

Referring to the power plant, Professor White pointed out that the huge lake upstream reduced the variation of upstream level almost to zero, yet paradoxically, the ten turbines were of the Kaplan type—costly machines designed to cope with a wide range of head and load. Would not one Kaplan have served the purpose, allowing the others to have fixed blades, possibly with greater efficiency?

Mr Josiah Eccles remarked that, as one who was interested in the earlier report to which Professor Naylor had referred, he had naturally followed the project with very considerable interest. He thought that the Authors would probably agree that it was in itself not a spectacular scheme in relation to size or to problems overcome, but to him it was important as a symbol of what could be done in a virgin country to develop the sources of power in nature. Most members of the Institution would be aware that the water-power resources of the world were very considerable indeed, and, if they were all properly harnessed, it would be possible to provide between two-thirds and three-quarters of the world's energy requirements by water power alone. Forty per cent of those resources were in Africa, and so the object-lesson provided by the Paper was one which those who believed in the future of science and in the development of a mechanized form of civilization would applaud.

In company with Professor White, he had been puzzled—and he hoped the Authors would provide an answer—why so little of the run-off of the catchment was being used for power purposes. To put it the other way round, the scheme had been developed in such a way that only a small proportion of the run-off would be used, or was capable of being used, for power purposes. He knew that it was necessary to walk before one could run, but it had seemed to him in the early days, and subsequently, that a larger view might have been taken of the problem, unless there was some countervailing argument against it.

He realized, of course, that account had to be taken of the evaporation from the reservoir itself. Some of that came back, but presumably it was already counted in the rainfall on the area before one started, so that it could not be taken twice over. If nature had been kind, what would really be a good idea would be to evaporate the tail-race. If it so happened that there was a big stretch of water downstream of the dam, evaporating that and redepositing it on the catchment area would be beneficial.

Mr Eccles remarked that in Table 2 the £10,000 surplus for 1962 was obtained after charging £25,000 of operation and management costs to capital. If that £25,000 was expenditure incurred on materials and labour used on capital works he had nothing further to say, but if it meant that certain renewal costs had been capitalized he thought that the 1962 position was a net deficit of £15,000.

Mr R. L. Chantrill observed that the catchment area was given in the Paper as 103,000 square miles, with an average rainfall of 43 inches per annum. He presumed that that referred to the catchment area which fed Lake Victoria. The Authors had then stated that 93 per cent was lost through various causes, leaving a utilizable balance of about 6·7 per cent. Presumably that meant that the climate as expressed by precipitation and run-off was a self-contained area, and that the water which was discharged from the lake was the balance between the precipitation local to the area and that which was brought in from the oceans outside Africa. He did not know whether that had been evaluated approximately, but if any figures were available, however rough and however largely guesswork they might be, he would be glad to be given an indication of them.

Looking to the future, he pointed out that the present power development scheme had been built across the river which discharged the waters down to the Sudan and Egypt. Those waters were essential for the very existence of the two countries downstream, and presumably there had been a very rigidly prepared agreement between the countries concerned to limit or control the rate of flow in the river. From the map in *Fig. 1*, it would be seen that downstream of Lake Victoria there were other lakes such as Lake Kioga and Lake Albert, all within the boundaries of the protectorate of Uganda, before the Nile finally left the protectorate to journey northwards to the Sudan and Egypt. Those two lakes, he imagined, formed quite an appreciable natural balancing reservoir, and, unless restricted by any form of agreement, he would expect them to be able to function as very appreciable balancing reservoirs between the flow required by power generation and the discharge of the river for irrigation in the Sudan and Egypt. In other words, provided that the total volume of water passed daily or weekly through Lake Victoria to Lake Albert was that required to maintain the discharge to the Sudan under the international agreement, the actual hour-to-hour flow could be varied to suit the power demand.

Admittedly that would not arise at the present time, but, looking to the future, it should have an important bearing on the power-generation costs, because, for approximately the same capital outlay on civil works—which, after all, represented the major item of cost in a project of the kind in question—the generation potential was increased roughly in inverse ratio to the daily or weekly load factor, and with that increase in the power potential of such a development the cost of power as such was reduced.

If the future development of the system was considered as an entity, it followed that the future Mutir dam shown in *Fig. 1*, with or without any power-generation plant, could be used to provide the controlled discharge northwards as required by the international agreement. That dam would presumably allow the river flow upstream to fluctuate with seasonal variations in the river flow as might be required for power generation at that time. Having the final regulation for irrigation discharge at the Mutir dam, he would expect the river system as a whole to be capable of sustaining a power capacity considerably in excess of the 2 million kilowatts mentioned by the Authors. Provided the river system and the developments were planned in that manner, he would hazard a rough estimate of the power potential, based on the load factor quoted in the Paper, as somewhere in the neighbourhood of 5 million kilowatts, with only a slight increase in the cost of the civil works.

While that did not arise now, the problem would become one of paramount importance as each successive generating station was built downstream of Owen Falls. Provision for that would not materially affect the cost of the civil works, apart from arranging the lay-out of the civil works to permit the installation later of additional generator units. As had been mentioned earlier, the present installation was restricted inevitably by the terms of the agreement with the countries downstream of Uganda, but, looking to the future, the ultimate development of the Victoria Nile basin within the boundaries of Uganda might be restricted in its power potential unless that was taken care of in the stations planned in the future downstream of Owen Falls. That was particularly important today, having regard to the indigenous power potential in relation to local mineral deposits in Uganda.

Mr J. F. Pain, on behalf of the Owen Falls Construction Company, expressed their very deep appreciation of the assistance which, as contractors, they had received both from the Uganda Electricity Board and from the consulting engineers, and for the very understanding way in which they had met the many difficulties which inevitably arose during the construction of a major work of the kind in question, particularly under conditions of rising costs and difficulties over delivery of materials such as were experienced in the post-war period.

⁂ **Mr. C. E. Andreae** referred to the statement made by Mr Paton, when introducing the Paper, to the effect that the average run-off from Lake Victoria was known to be approximately 21,000 cusecs but that all else was largely conjecture. If, however, Lake Victoria was considered alone and the average rainfall was assumed to be 43 inches, it was improbable that evaporation losses would exceed about 9 inches in the year, so that the net fall would be 34 inches, or about 65,000 cusecs. Since that was three times the measured run-off, even without taking into account the remaining 77,000 square miles of catchment area, had the Authors any theory to account for such a remarkably small run-off ? Was there, for instance, any evidence to suggest that some of the lake water might drain away at a lower level and find its way to the Nile farther downstream ?

Mr T. A. L. Paton, in reply, said that the Authors were very grateful to Lord Reith for his kind remarks about the scheme.

Mr Haldane had raised the question of what advantage could be taken of the increased discharge of the Nile if the swamps round Lake Victoria were drained. The answer was that much greater development of the agricultural resources of the East African territories would be necessary in the future and a survey of the water resources of Uganda, including the drainage of swamps, had now commenced. Certain areas in Uganda were short of water and, in order to grow crops, irrigation was essential to augment the rainfall; by no means did the whole of Uganda have an average of 43 inches of rain per annum.

On the question of the building-up of storage, according to the records, which had been kept for about 52 years, it might take anything between 8 and 17 years to build up the additional storage in Lake Victoria, assuming an average discharge of 505 cumecs. It might in fact take even longer, for during that time Egypt expected that there would be occasions when the combined flow of the Blue and White Niles would be less than her requirements, which would entail increasing the average discharge for a period. The negotiations with Egypt had not yet been settled regarding

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⁂ This contribution was submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

payment of compensation for the use of Lake Victoria for storage purposes.

The Authors thanked Mr J. K. Hunter for his contribution to the discussion.

Professor Naylor had made some very interesting comments on the earlier history of the scheme, which was well understood and much appreciated. Possibly in post-war days the remarks of General Smuts had been slightly countered by the effect of the hydro-electric schemes in the north of Scotland. There, far from spoiling the countryside and disturbing the "rustic simplicity" of the Scots, the developments had improved the appearance of the country and were generally agreed to be an asset to it. Incidentally, the remark had been made that Lake Victoria was the size of Scotland, but he would like to point out that it was also the size of Ireland.

With regard to reducing the flow of the Nile, first of all in a temporary way during the construction stage, the Authors had hoped for permission to restrict the flow much more substantially than in fact had been done, but the agreement with Egypt and with the Sudan prevented that being done, and the Authors had had to keep very rigidly to a certain minimum flow. The permanent restriction of flow is in the hands of Her Majesty's Government and the Government of Egypt.

Professor Naylor had referred to the relative costs and advantages of locating the dam farther upstream. That had been carefully studied by the Authors before selection of the present site. The main reason for the change had been to obtain adequate space for ten sets, each of 15,000 kilowatts, without excessive cost for the head-race or for the power house and tail-race. The lower site was much more suitable in that respect. So far as the dam was concerned there was a difference in foundation level of only about 3 feet; in each case the foundations would have been formed on a hard ridge of amphibolite rock. There was just as much risk of finding fissures and pockets of gravel in the rock in the case of the upper site as in the lower site, the river conditions being almost identical. The only possible advantage in the upper site would have been in the temporary works to control the river. But the Authors considered that such saving in cost would have been more than outweighed by the other disadvantages. On the question of the design of the dam the radius was too great to take advantage of the arch effect in the calculations. The curve had followed a hard amphibolite rock ridge forming the lower falls.

Mr Harding had referred to disturbing the "rustic simplicity" of the Africans. The Colonial officials had very much in mind the importance of developing agriculture in the future as well as industry, which would be attracted by the power development.

Professor White and Mr Eccles had asked what really happened to the water that fell on the lake. The difficulty was that there was only one clear factor—the volume of water flowing down the river, which had been recorded for a number of years. Even that factor would now be checked by calibrating the sluices and the turbine intakes. The rest of the hydrological data was somewhat conjectural, depending on only a few rain-gauges and rather inadequate information.

The survey of the water resources of Uganda which was now proceeding would enable a more detailed study to be made of the reasons for the small run-off from the Lake catchment—a point raised by Mr Andreae. Undoubtedly much water was lost in the swamps bordering the Lake and in the lower reaches of the rivers feeding the Lake. There was moreover no evidence that the lake water drained away at a lower level, thus finding its way into another river valley or the Nile farther downstream.

Another point mentioned by Professor White was the action of the jets of water from the sluices in carving a hole for themselves downstream. The Authors had recently received a report from the site saying that the levels of the rock taken immediately downstream of the dam had shown an increase in height. Rock was being washed up against the back face of the dam, which was one of the objects of the design of bucket, and the result of very careful experiments by Professor White and his assistants at Imperial College.

Mr Eccles had rightly said that the Owen Falls scheme was not, from an engineering point of view, a spectacular achievement. Many higher dams and bigger power stations had been built. The Authors felt, however, that particularly from the construction side, the work had proceeded very smoothly, very nearly up to time, with day and night operation.

Mr Chantrill had questioned the figure of 2 million kilowatts of power which might be developed down the Nile to Lake Albert. That output had been calculated on the basis of firm power, not with 60-per-cent load factor, but it could be increased. There were, however, a number of difficulties, such as the fact that Lake Kioga was a big marshy lake, where about 12 per cent of the flow was lost.

Mr Andrae had referred to the amount of the evaporation losses from the catchment area. The calculated average evaporation over the land catchment area, based on the reports from eighteen meteorological stations of the East Africa Meteorological Department, worked out at approximately 44 inches per year. The amount of evaporation over the Lake area was, however, not known with any accuracy, but the total evaporation of both land and water area would be of the order of 40 inches per annum.

Mr R. W. Mountain, who also replied, said that Mr Haldane had referred to the provision which possibly should be made for a larger installed capacity; there was no provision for any increase in the installed capacity at Owen Falls of 150,000 kilowatts, but the intention was to look farther downstream for additional capacity. He thanked Mr Anscombe for the additional information which he had given on the electrical equipment. In reference to the proposal by Professor White that one or two Kaplan sets should have been put in and the remainder should have been of the simple propellor type, he believed that the fall-off in the efficiency curve of the propellor-type turbine, with variations in head as well as variations in load, justified the installation of all Kaplan sets for the Owen Falls station.

Sir Charles Westlake replied (in writing) to the point raised by Mr Eccles on the transfer of certain sums to capital account, and stated that those sums were properly chargeable to capital.

Correspondence

Mr A. E. Griffin observed that the Owen Falls dam was strikingly different in character from any other hydraulic structure on the Nile and its designers certainly could not be accused of suffering from the pyramid complex. It was true, of course, that the project presented a different type of problem to that normally encountered and the Nile in that area was young with simple habits. But the scheme had been dealt with strictly on its merits and the final answer, as depicted by the artist, was no eyesore.

Mr Griffin had pictured a long and graceful spillway dealing with the surplus flow. It would have made a noble spectacle, especially if floodlit by night. Nevertheless, the 10-foot range of water level and other features revealed in the Paper, clearly made the bucket sluice appear a better proposition. Undoubtedly, conditions were very suitable for the application of the ski-bucket principle ; that might have been the decisive factor in adopting sluices rather than a spillway.

It was stated in Appendix I that the model tests revealed a coefficient of discharge of 0·828 for a single sluice and that the pressure plotting disclosed the amount by which the upturned toe reduced the discharge. With a conventional sluice of those dimensions, fully open, running free under a good head, the coefficient in the usual formula * might be about 0·9 in view of the fairly generous bellmouth. The actual coefficient of 0·828 would mean that the coefficient resulting from the bucket would be 0·92 ; in other words, the upturned toe reduced the discharge by about 8 per cent under the specified condition.

When endeavouring to solve discharge and jet problems by pure calculation and the employment of the usual formulae, one usually assumed absolute water whereas there was probably a good deal of aeration which must have an influence on the discharge and the whole character of flow, no matter how carefully and correctly the coefficients had been chosen. It was, therefore, logical to enquire if the piezometers disclosed the longitudinal water surface throughout the length of the tunnel (assuming that it was not visible) and consequently made possible the calculation of the percentage of air entrainment under various conditions. Although that would not be important at Owen Falls, it might be a factor of some account in other circumstances in connexion with the dissipation of energy and the passing of a very large discharge at a specified upstream level.

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* $Q = C.A. \sqrt{2g.\,(\text{head on sill} - 0{\cdot}9 \text{ of opening when fully open.})}$

It was stated that the gates normally would be fully open. That implied that regulation would be effected by varying the number of vents and, consequently, never reducing the intensity of impact. Even if the risk of unsymmetrical flow downstream did not matter in the case in question, it would be interesting to learn what were the objections to gate regulation and spreading the impact uniformly, considering that the maximum head was not in excess of that under which slide gates could be operated successfully ; in fact, the conditions were about the same as at Aswan, except for the greater altitude which might give a slightly higher tendency to cavitation of which, however, it was stated that there was no undue risk. As a superficial observation, it seemed that nine or even twelve narrower vents, gate-regulated, would have produced kinder downstream conditions though, perhaps, not quite so efficient from the hydraulic aspect. With regard to what happened after the impact, it was not quite clear to Mr Griffin whether the hard rock-bed had been left in its natural state or if a scour pool had actually been excavated in advance to reduce surface turbulence.

An interesting feature was the use of a reinforced-concrete surround for the tunnels in place of the Nile custom of ashlar, which was expensive and full of troublesome joints. He wondered if the designers would have used concrete if the water had been silt laden, as in the lower river (since that appeared to be the only reason for using ashlar), or if they would have resorted to steel or wrought iron, having regard to the rigorous hydraulic conditions under which the sluices would work. It would be interesting to know what degree of wear was anticipated despite the water being relatively clear. Apart from its resistance to abrasion, ashlar seemed to compare very unfavourably with concrete but he could not recall seeing any conclusive literature on the behaviour of concrete sluice lining under ordinary Nile conditions. Such a study, if it had not been done already, would readily lend itself to laboratory investigation.

Mr Frank Grundy, of Nairobi, observed that in the Section of the Paper which dealt with water power potential, the Authors had given some information about the hydrology of Lake Victoria, and had referred to the importance of increased storage in the Lake. Change of storage was calculated from the area of the Lake surface and the mean change in water level. The necessity of measuring the mean change in water level with great accuracy was evident from the fact that a change of only 1 inch represented a volume of water equal to a discharge of nearly 2,000 cusecs for one year.

The recording of mean change of water level was complicated by variations in hydrological and meteorological conditions over the Lake area.

Using very approximate values of rainfall and evaporation, based on the limited information available, it appeared that in the northern half of the Lake, the average annual rainfall exceeded the corresponding evaporation by about 3 feet, whilst in the southern half, evaporation exceeded rainfall by about 2 feet. Furthermore, inflow from rivers in the northern part, including the discharge of the Kagera River, greatly exceeded the inflow to the southern part. It therefore followed that there was a slow movement of water from north to south, the average annual volume of which probably exceeded the volume discharged at Jinga.

Those differences in rainfall and evaporation, which were subject to seasonal and secular variations, surely resulted in oscillations in the level of the Lake surface, whilst further oscillations would result from wind effects. Therefore in order to avoid serious errors in calculating changes in Lake storage, a number of gauges were required around the Lake and on some of the islands, each provided with a water-level recorder.

The Authors, in reply to Mr Griffin's comment on the flood sluices for discharging the flow of the Nile through the dam stated that the behaviour of a single sluice had been observed with a transparent plastic model but, since the model velocities were much lower than those in the full-size structure, air entrainment had not been simulated. The model did, however, to some extent indicate whether conditions likely to cause air entrainment would occur in practice. Very slight vortices were in fact formed on the water surface immediately above the sluices when the Owen Falls scheme was operated at low head. However, it was expected that those vortices would disappear when the head was raised another 20 feet to full height.

With regard to the regulation of the flood gates, they were designed to operate at partial flow. But the wear and maintenance on the gates was less if they could be operated either fully shut or fully open, and, with the reasonably constant flow of the Nile, it was anticipated that operation on that basis could be carried out without any difficulty.

No scour pool had been formed artificially where the jets from the sluices discharged on the rock bed. In the model experiments some slight improvement had been observed by deepening the bed at that point but only two sluices discharged on to fairly shallow water and the Authors felt that nature might be given the opportunity of forming the scour pool.

High-quality reinforced concrete had been used to line the tunnels and buckets of the flood sluices, whilst a small amount of metal protection had been provided immediately below the critical parts of the gate grooves. There might in time be some wear on the concrete but construction joints had been eliminated as far as possible and the workmanship was of a high standard. Since access was possible for maintenance purposes, elaborate and costly provisions for eliminating wear, which were not always successful, had not been incorporated in the design.

The oscillation of the Lake level mentioned by Mr Grundy was an interesting phenomenon. The Authors entirely agreed that more water-level recorders were necessary and desirable round the Lake shore to forecast the amount of water which should be discharged at Owen Falls when the Lake was controlled for storage purposes.

Correspondence on the foregoing Paper is now closed.—Sec. I.C.E.

SUPPLEMENTARY MEETING

26 May, 1954

WILFRID PHILIP SHEPHERD-BARRON, M.C., T.D., President, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Institution were accorded to the Authors.

Paper No. 6008

"Owen Falls: Constructional Problems"

by

Dennis Percy Bertlin, M.Eng., M.I.C.E., and
Henry Olivier, C.M.G., M.Sc., Ph.D., M.I.C.E.

SYNOPSIS

Completion of the Owen Falls multi-purpose scheme, situated in Uganda at the head of the White Nile and designed to control Lake Victoria, marks an important stage in the plan eventually to utilize the full hydro-potential of the Nile by means of a series of schemes to be located in four countries—Uganda, Sudan, Ethiopia, and Egypt.

This Paper deals with the constructional problems relating to the civil engineering contracts, including the design of temporary works. An account is given of the methods of water control during construction, of excavation and the production of in-situ and precast concrete. Brief descriptions supported by statistics, are given covering the lay-out of services and plant, control of materials, accommodation, and labour, with special reference to environment and health.

For further information on the subject reference should be made to the Papers on: (1) the general historical, economic, and design aspects of the scheme; and (2) the constructional problems relating to the electrical and mechanical contracts (a Paper published by the Institution of Electrical Engineers).

INTRODUCTION

THE consulting engineers' report on the proposed scheme was presented to the Government in early 1948, and instructions to proceed with the work were given in June 1948. As a result of the additional site surveys, boring, and model experiments, the Uganda Electricity Board decided to extend the capacity of the power station to ten sets of 15 megawatts capacity, and the final scheme was therefore re-located about a thousand feet below the original site chosen at the Owen Falls (see Fig. 1, Plate 1).

Tenders were invited during the middle of 1949, and the main civil engineering contract was awarded in September 1949 to an international consortium, the Owen Falls Construction Company (see Appendix I), whose representative arrived on site in November 1949. The original contract was on a unit-rate basis for a duration of 4 years. Erection of offices and temporary works commenced early in 1950, at the same time as excavation in the power-house area. The first major pour of concrete was in the power-house foundations in May 1951, and concreting commenced in the main dam during September 1951. Water was raised behind the dam in December 1953, and the first generator was commissioned during January 1954, the remaining three sets under the original order having been scheduled to be commissioned at intervals of approximately 4 months. Sets 5 and 6, ordered after the date of letting the main contract, are scheduled to be commissioned during 1956.

Progress with regard to the different sections, up to the end of April 1954, is shown in Fig. 2, Plate 1. The graph shows the build-up of labour. The best concreting rate achieved was 13,500 cubic yards placed during August 1953. The labour force reached a peak figure about October 1953, when the African employees totalled 2,500, the European and Asian employees 300, contractor's staff 47, and over the peak period the consulting engineer's site supervisory staff, including inspectors, totalled 25.

During the tenure of the contract, the Board sanctioned extra works amounting to about £650,000 in value, and in January 1951 an Acceleration Programme, incorporating a bonus clause, was approved which assisted the contractors to operate on night shifts for a period exceeding 3 years, thereby reducing the contract times materially.

Construction difficulties may be classified under three broad headings: water control and cofferdam construction, having regard to the unpredictable nature of foundation conditions in the river bed; labour problems; and the phasing of deliveries of construction materials in the difficult post-war years, having regard to the long hauls involved. The total tonnage of temporary and permanent materials brought to site by East African Railways and Harbours, from Mombasa, a distance of more than 750 miles, amounted to about 80,000 tons, of which 36,000 tons represented cement from Europe.

SURVEYS

The Project Report was based on a preliminary survey by the Uganda Public Works Department. Final location surveys were made over the period March–September 1948, when more than 3,000 lineal feet of core drilling was carried out and about 800 soundings taken in turbulent water, using wires tensioned across the river by means of winches on each bank. The sounding cable was positioned by simultaneous readings on two theodolites and by direct level readings. The survey parties were coordinated by using portable radio sets.

It was necessary to carry out soundings at Jinja Bridge during April 1951 in connexion with the design and construction of concrete protection works to the portions of the steel arch carrying the road deck which would be submerged after water raising. At this point the water narrows to a width of approximately 250 feet and a submerged rock barrier runs obliquely across the river, the measured velocities at that time being 24 feet per second. It was calculated that velocities under the bridge would not exceed 4 feet per second during periods of maximum discharge from the lake after water was raised to operating levels.

A considerable amount of exploratory work was carried out in connexion with the final design of the cut-off walls at each end of the gravity dam. At the time tenders were invited in 1949 one borehole had been sunk in the cut-off area on the west bank and two on the east bank. During September and October 1950, three more bores were sunk on the east bank. Generally the classification of the rock is :—amphibolite with clean joints ; amphibolite with stained joints ; broken amphibolite ; decomposed amphibolite ; soil, clay, etc. The geological interpretation is that weathering, caused by percolating water carrying vegetable juices, proceeds downwards. It follows that any classification can only be arbitrary in that each type merges slowly into the next, and there are no clear-cut dividing lines between any two classes of rock. In practice, not only the recovery, but the interpretation of bores proved difficult. Tests were started in October 1950 to determine whether the overlying material was impermeable or could be made so by grouting. In addition, through the assistance of the Director of Geological Survey, Uganda, a gravimetric survey was carried out for both banks early in 1951. The location of additional boreholes and areas covered by gravimetric survey are indicated in Figs 3, Plate 1, together with typical logs for boreholes on both banks. It was concluded that the material overlying solid rock in these regions could be regarded as practically impermeable, and that the banks could be treated as natural earth dams. Short concrete walls, 5 feet thick, were carried into the banks for a distance of 15 feet from the ends of the gravity dam. Radial holes were drilled for pressure grouting to form a continuation of the grout curtain under the main dam cut-off which is described elsewhere.

CLIMATE

Climate had to be considered at the outset from the human as well as from the material angle. Regarding the former, the core of the artisan force consisted of men recruited in European countries : Britain, Denmark, Holland, and Italy. Jinja is located at an altitude of 3,878 feet above sea level. The average temperature is 73° F. with an annual variation of 3·8° F., 74·7° F. in February and 70·9° F. in July. Mean maximum temperature recorded occurs in January (84·7° F.) and the mean minimum temperature in August (61·9° F.). Mean annual relative humidities are 82 per cent at 08.30 hours and 60 per cent at 14.30 hours, with ranges of 11 and 15 per cent respectively. At night, however, the humidity approaches 100 per cent nearly all the year round. Precautions were taken therefore against the cumulative effects of tedium resulting from uniformity of climate, and excellent recreation facilities and accommodation were provided as mentioned later.

From the material point of view, the climate is ideal for all-year-round concreting, though curing could not be neglected during day-time hours. The greatest factor was rainfall, which in Uganda occurs all year round with two peak periods : the " long " rains (March–May) and the " short " rains (August–September). The wettest month is generally April with a highest recorded rainfall of more than 11 inches. After the commencement of the contract, it was discovered that a rate of 12 inches to the hour, falling for four minutes, had been recorded at Namulongi Cotton Research Station, near Kampala. It was therefore necessary to provide hardcore bases under all temporary access railways and for most access roads, totalling 5 and 3½ miles respectively, and to provide liberal site drains to ensure continuity of work. Whereas man hours were lost because of rain during the first half of 1951, once those measures were taken, time so lost became negligible.

HEALTH PESTS AND DISEASES

It was known at the outset that Jinja was plagued with a pest known as Mbwa fly (Simulium damnosum), which breeds in the white water of rapids, and the bite of which is extremely irritating. In some cases blindness is caused (onchocerciasis). This pest was almost completely eradicated after the Medical Department of Uganda had applied doses of D.D.T. to the Nile at Ripon Falls during 1952.

Malaria was known to be common, but the doctor employed by the Owen Falls Construction Company succeeded to a large extent in controlling the incidence of this disease by prophylactic measures, though a bad malarial outbreak was experienced in 1951. In succeeding years the number of cases reported dropped considerably; that was probably because a larger area of bush had been brought under control and a regular issue of anti-malarial drugs had been made to Africans since early 1952. The tsetse forest came close to the African camp on the west bank of the river, and strict measures were taken to ensure clean belts around the Camp and to keep grass and undergrowth down within its boundaries. The African camp was provided with a hospital, and a clinic was maintained for Europeans.

Throughout the contract period, nuisance was caused during night shifts by heavy swarms of Lake (May) fly, which are attracted by lights. Although these insects did not bite they were a nuisance to workmen, foul fishy-smelling messes were left under lamp posts where swarms had died, and their remains on painted surfaces (except cellulose paints) proved corrosive in the long run.

The main lesson learnt has been that rigorous medical examinations for men who are to be employed in the tropics, are in the interests of both the employer and employee.

ACCOMMODATION

To ease the organization problems of the contractors, the Uganda Electricity Board made arrangements in advance of letting the main contract for the construction of camps for 1,500 African labourers and for building flats and a recreational centre for staff. The contractor increased this accommodation by providing a camp for staff and artisans, houses for senior staff, married quarters for a proportion of Africans, as well as seventy-six bungalows for artisans, complete with electrical equipment.

The general lay-out of accommodation relative to the scheme, is shown in Fig. 1, Plate 1. Amenities in the African camp included a church, hospital, canteen, shops, cinema, and sports grounds as well as vegetable plots (shambas) for cultivation by families.

LABOUR

The growth of the labour force is illustrated in Fig. 2, Plate 1. African, Asian, and European labour worked together with great flexibility. All the unskilled work and much of the semi-skilled was done by Africans drawn from about seventy-five different tribes, mostly from up-country Uganda, though a number of Kenya (mostly Luo), Sudanese, and Congo boys found their way to Owen Falls. Some of them attained skill as mechanics, blacksmiths, plant operators, and steel erectors. Most of the skilled work, however, was done by Europeans, whilst more than a hundred Asian (predominantly Indian) carpenters, mechanics, masons, and clerks, completed the labour force.

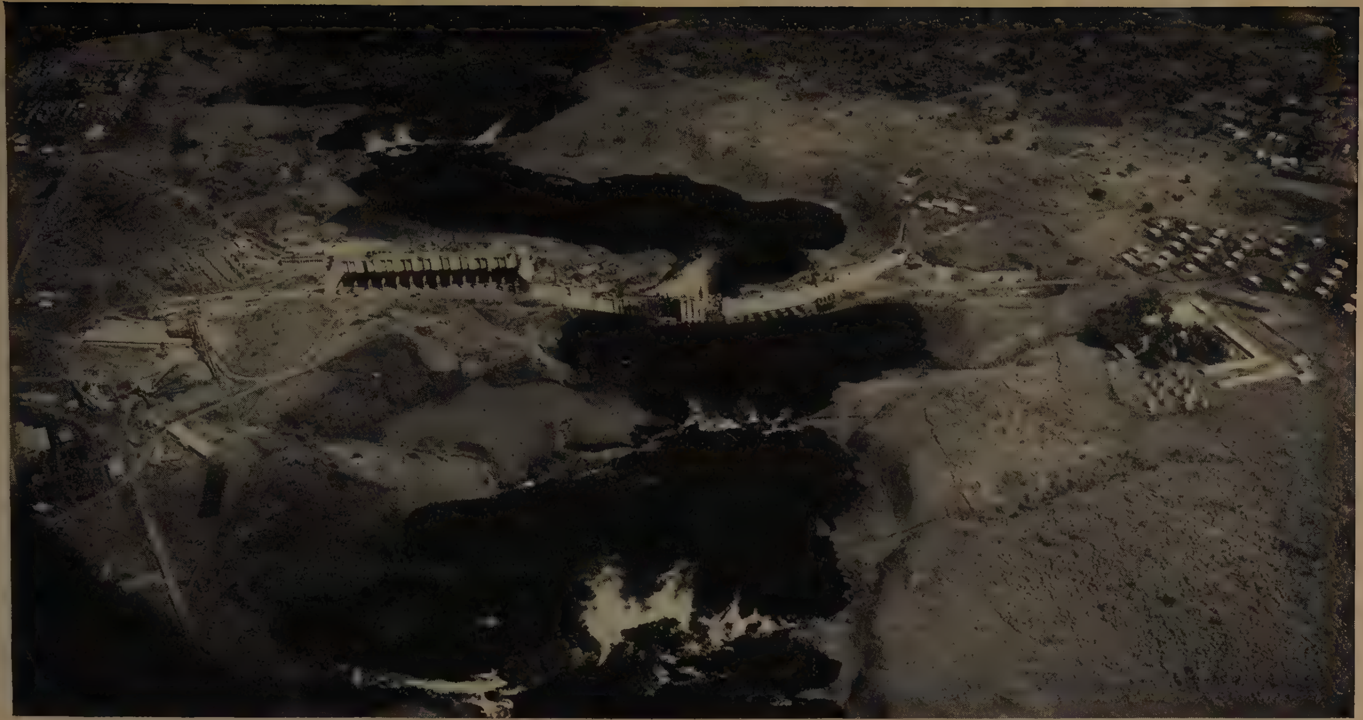
CONTROL OF WATER

Hydrology, Method of Control, and Stages in Construction

During the past 50 years the discharge of the Nile from Lake Victoria has varied between 10,500 and 43,000 cusecs. For the period of construction it was agreed that flow might be restricted to 21,100 cusecs, such a control having little influence on the level of the Lake because of its large area of 26,000 square miles.

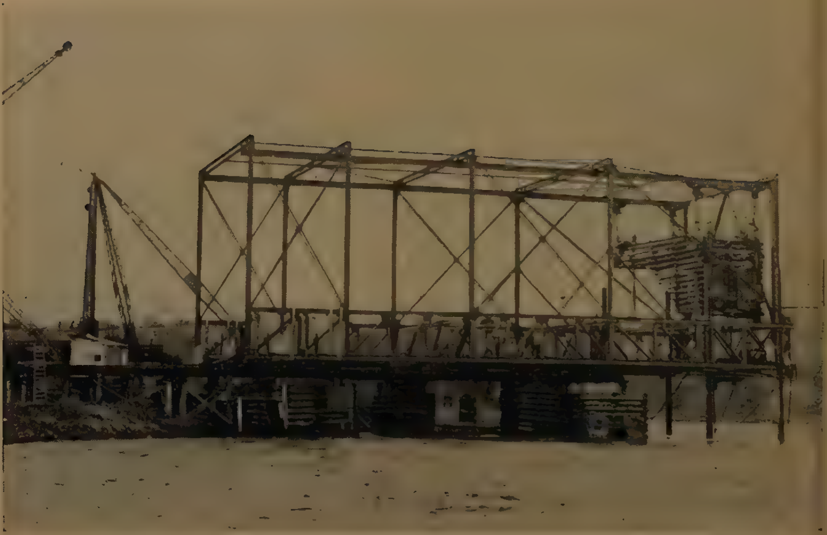
Various alternatives were considered for cofferdamming the river and for its diversion during the construction period, first by the civil consultants and subsequently by the contractors. The maintaining of the flow of 21,100 cusecs, combined with the nature of the river bottom, made this an interesting problem. One of the methods considered was to build temporary sluices through the right bank of the river and to form cofferdams by means of gabions on a line more or less following the Owen Falls themselves. That would have involved cofferdamming a very much bigger area than was in fact done, which had certain advantages. However, the heavy cost of the construction of the sluices on the right bank to

Fig. 9



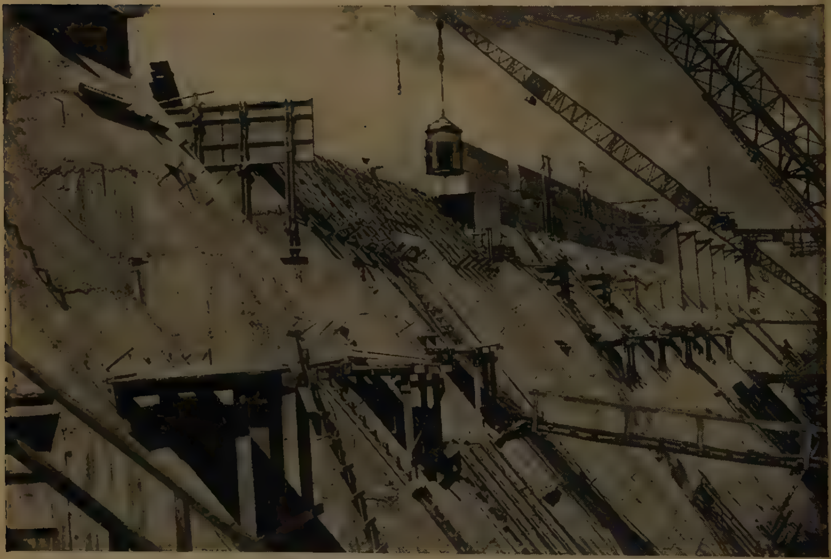
Aerial View of Site during Later Stages of Construction

Fig. 10



Traveller about to place a Crib for Cofferdam

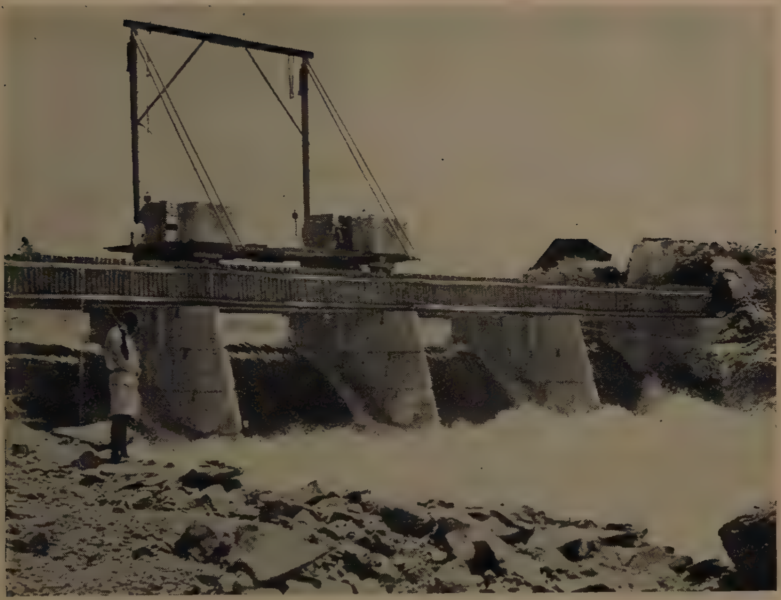
Fig. 11



Concreting the Downstream Face of the Dam

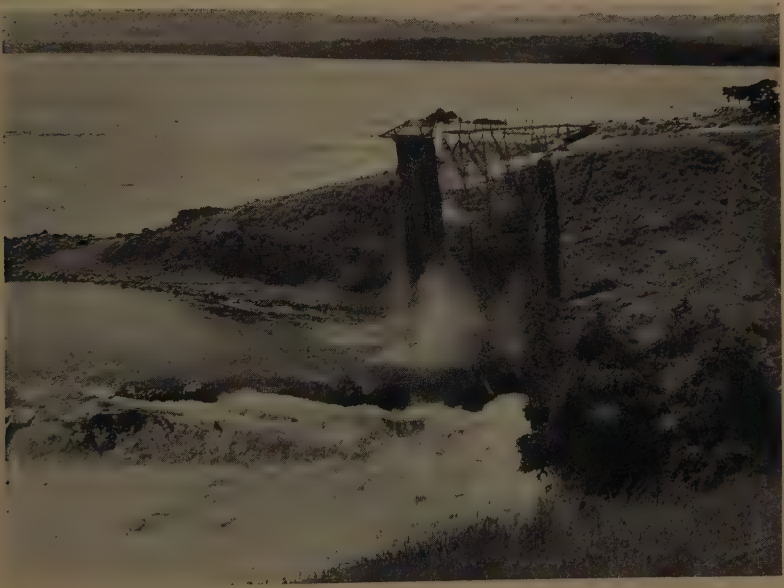
THE PRECAST COLUMNS FOR CARRYING THE ROADWAY ARE BEING PLACED
THE GAPS USED FOR WATER DIVERSION CAN ALSO BE SEEN

Fig. 12



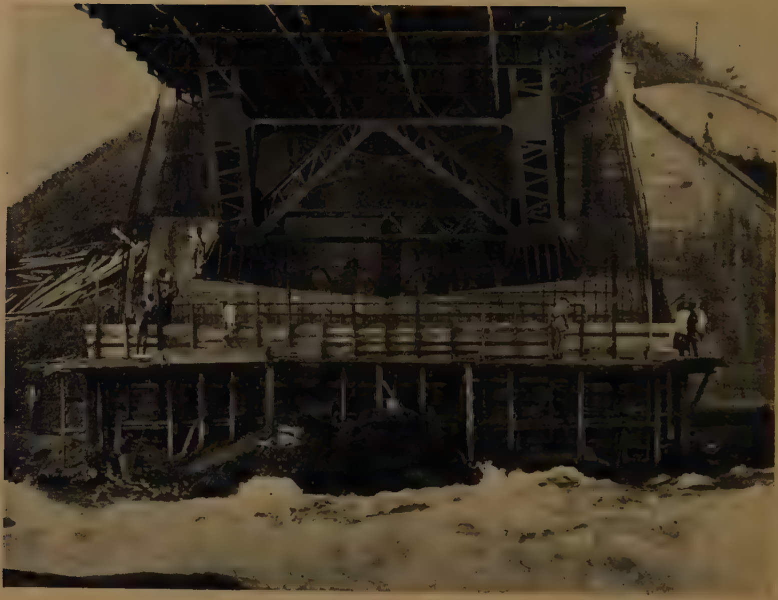
Ripon Falls Temporary Sluices

Fig. 13



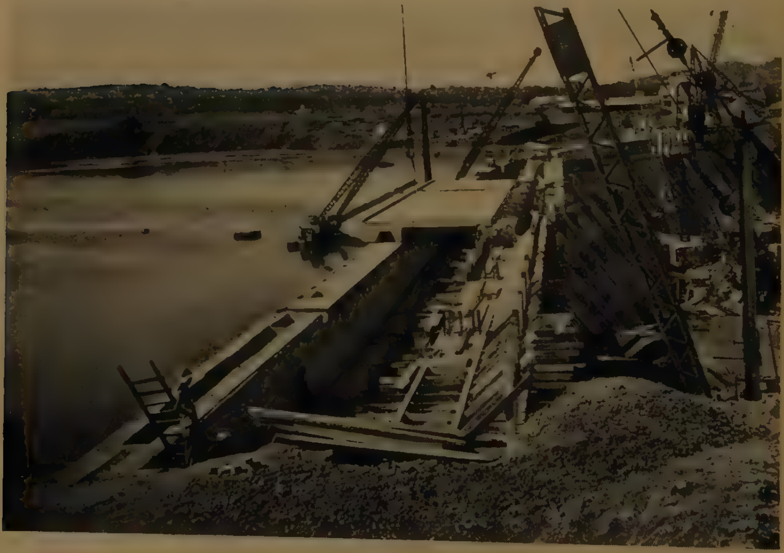
Base of Tower Blasted and Tower Falling to Dam the River at Ripon Falls

Fig. 14



Construction of the Umbrella to Protect the Hinges of the East Abutment of Jinja Bridge

Fig. 15



General View of Precast Slabs and In-Situ Beam Shuttering for Main Dam take such a large flow at a small head gave the advantage to the crib scheme which was eventually adopted.

Physical control was provided at Ripon Falls at the outlet of the Lake, where two islands divide the river into three channels. About half the river flowed through the west branch, and by blocking this channel and substituting for it a series of sluices on the east bank, it was possible to control the flow, and all cofferdams were designed for a water level corresponding to 21,100 cusecs, thereby making a substantial saving in the net cost of the temporary works.

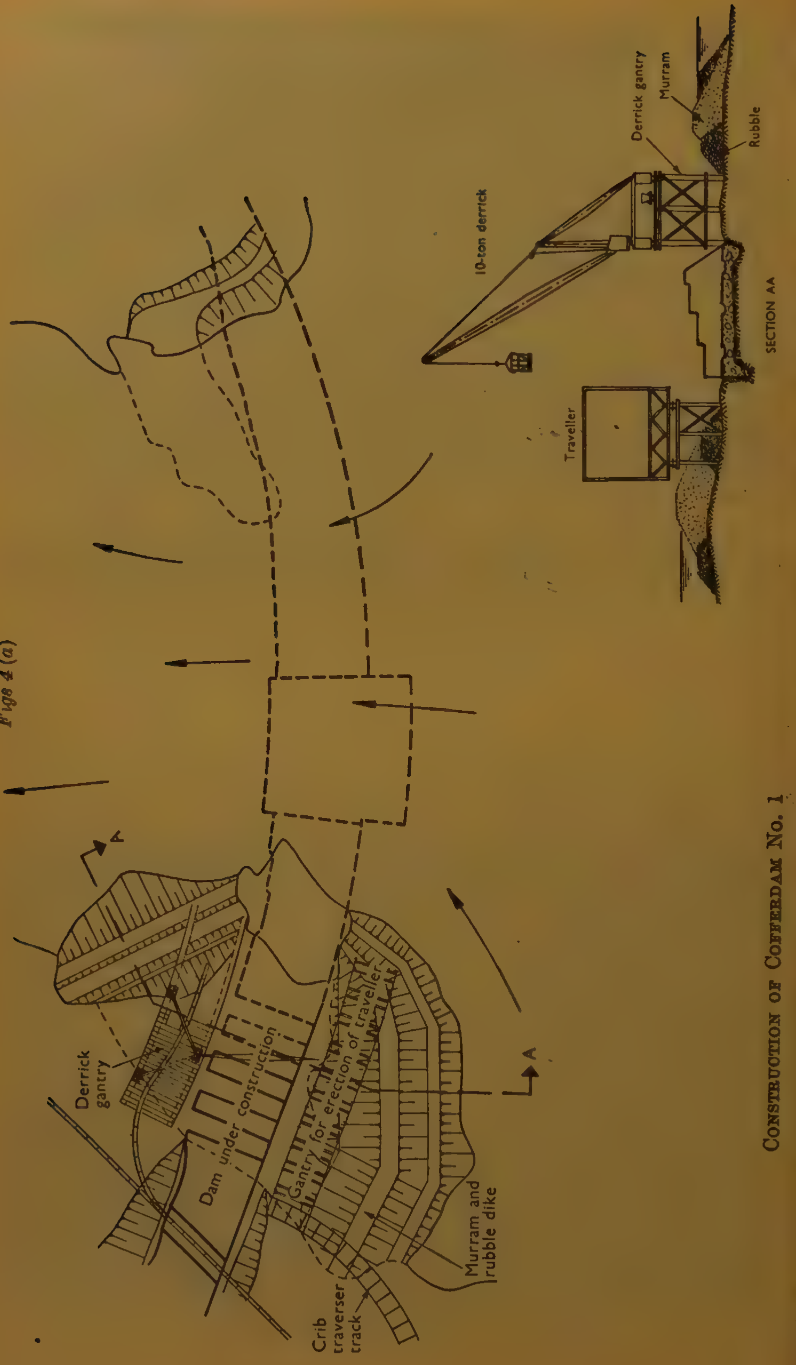
By agreement with Egypt, whose representative was stationed at Jinja, weekly current-meter readings were taken at Namasagali, about 55 miles downstream from Ripon Falls, and the mean of four such readings constituted the official monthly discharge. The period of control specified was 24 months, and by December 1953, shortly before control ceased, the Lake had ponded about 4½ inches, representing a storage of 290,000 million cubic feet (6·7 million acre-feet) available to Egypt when run-of-river conditions were re-introduced.

At the site of the main dam across the Nile, three cofferdams were constructed (*Figs 4*). The first was formed by two dykes joining the west bank to an island on the line of the dam. Whilst this work was going on, a spit of land jutting out from the east bank was removed to allow a greater discharge on this side of the river. The second cofferdam was then formed as an extension to the first. It extended sufficiently to the east to permit two-thirds of the sluice dam to be built within it. Whilst the permanent work was under construction in the first two cofferdams, cribs forming the upstream arm of the last cofferdam were placed. The river was then diverted through gaps left in the main dam west of the sluice dam, after which the last cofferdam was completed. In order to assist in the design of the cofferdams, it was decided to construct a hydraulic model, to a natural scale of 1 : 100, which was made in the Hawksley Hydraulic Laboratory of Imperial College, London. Owing to the difficulty of obtaining accurate soundings in such a deep fast-running river, it was often not possible to reproduce the bed of the river with accuracy. However, by using photographs and sketches of the flow, and by checking the water levels with those measured on site, it became evident that no serious error would be made. The main purpose of the tests was to check the water levels in the various stages of construction, for those levels controlled the design of the cofferdams. The water levels in model experiments were measured by a vernier manometer, which gave results to ±1 millimetre. Comparing the test and site readings before and after obstructing the flow, showed that the model had given reliable results.

Ripon Falls Weir

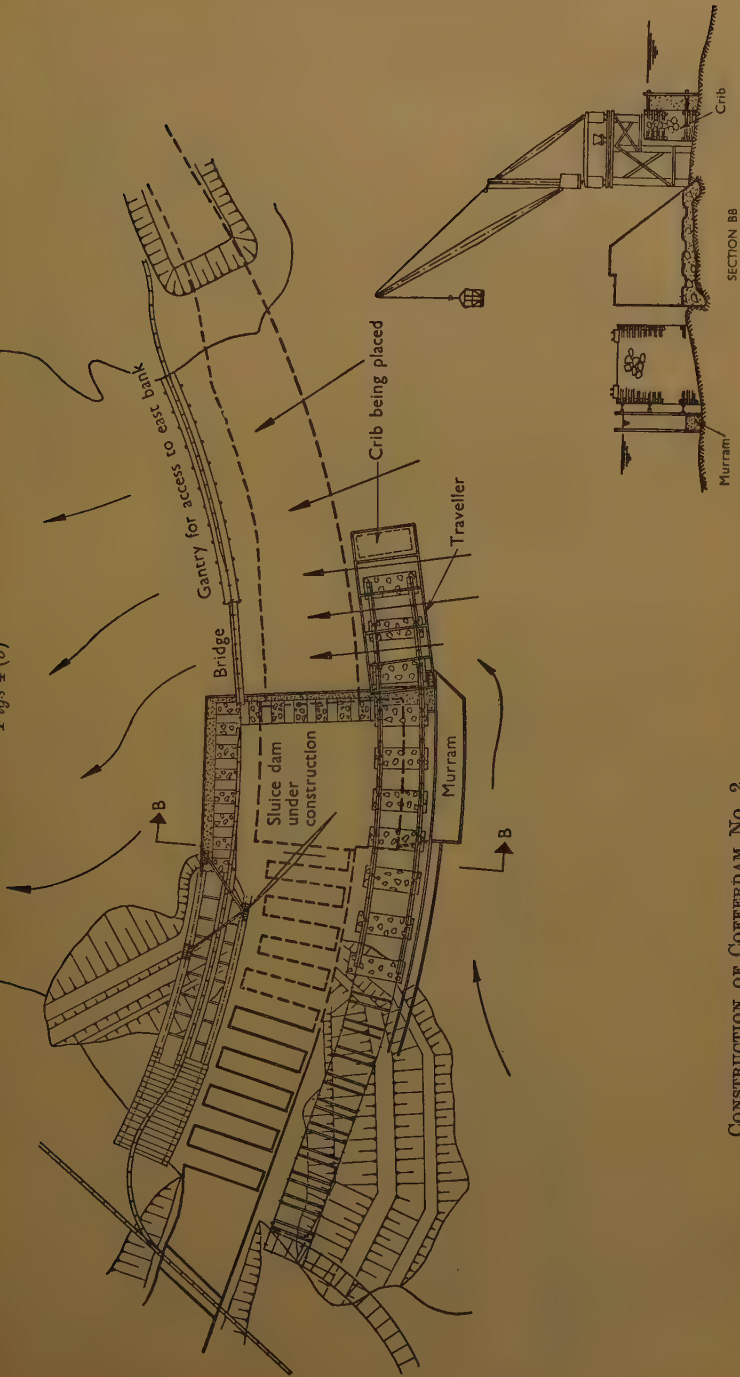
The weir at Ripon Falls consists of piers constructed of precast concrete blocks, excavation for which was carried out in the dry within a spit

Figs 4 (a)



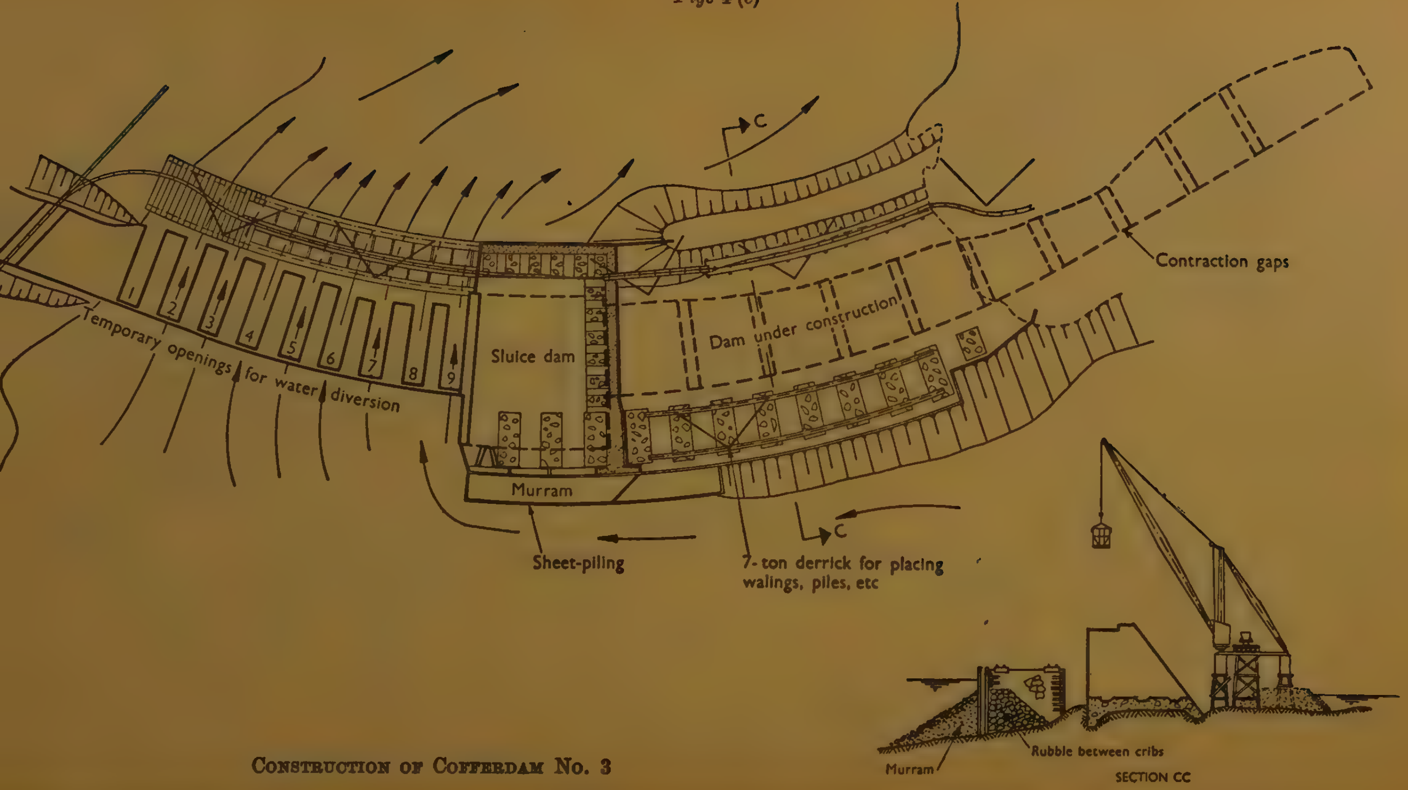
CONSTRUCTION OF COFFERDAM No. 1

Figs 4 (b)



CONSTRUCTION OF COFFERDAM NO. 2

Figs 4 (c)



Construction of Cofferdam No. 3

of land forming the eastern flank of the Falls, with stoplogs fabricated from sheet-piles. Additional weight was provided by an access bridge, spanning the piers, made of sheet piles and rubble ballast. Stoplogs were operated by means of a rig mounted on the bridge to traverse across the sluices. Whereas 10-ton hand winches were used for raising the stoplogs, lowering was achieved by the simple and effective method of tapping the end of the stoplogs with a vertical rail, operated manually.

The west channel was blocked by first dropping across it a rockfilled tower, 70 feet high by 10 feet by 10 feet, made from eucalyptus poles and torpedo netting, felled by blasting the timber supports on the river side, then following up by tipping rock behind it, using dumpers. As an alternative, the use of rock gabions was considered, but it was decided that the method adopted would be less costly and equally efficient.

Method of Constructing Cofferdams (*Figs 4*)

Cofferdam No. 1.—The two dikes forming cofferdam No. 1 were constructed by end-tipping rubble banks, using murram (laterite) as a seal. Protection of the murram from the erosion of the river was provided by cane mats pegged on the slope of the bank or by layers of rubble. The areas over which the murram seal was deposited were cleaned of loose deposits by dragline and grab.

Cofferdam No. 2.—The second cofferdam consisted of rockfilled timber cribs, prefabricated in sections not exceeding 45 tons in weight, which supported heavy steel beams placed at two levels; against these beams were driven Dorman Long Krupp Mark III sheet-piles purchased by the Uganda Electricity Board before the contract had been awarded. Crib sections, constructed by a 6-ton travelling steam crane and transported to the cofferdam on bogies and a traverser which negotiated a curve of constant radius, were made from eucalyptus poles not exceeding 41 feet in length, brought from Kaptagat, Kenya. Poles of sufficient size were not available in Uganda. The ends of all poles were bound with wire to prevent splitting, and the minimum top diameters specified were 8 inches for top poles and 10 inches for bottom poles.

A structural-steel "traveller," carried on bogies and operated by hand winches and tackle, was used to place the upstream cribs, which were 41 feet by 16 feet in plan, and were spaced at 32-foot centres. The cribs were placed in sections, usually four per crib; the bottom section was tailor-made to fit the contours of the river bed, which were determined from soundings taken by a cantilever arrangement fitted at the forward end of the traveller. (Figs 5, Plate 2.)

The crib section, lifted at the rear end of the traveller by four 15-ton chain blocks travelling on two joists running the length of the structure and supported by its framework, was moved forward until vertically over its destined position. It was then lowered and wedged into a frame which travelled vertically on four members attached to the traveller framework, and which were capable of being lowered or raised to suit the depth of the river (though they were never allowed to rest on the bottom). The two downstream members were supported by a diagonal 1⅛-inch wire rope, which transmitted to the main structure the force of the current on the crib section whilst being lowered, the traveller having been anchored to the two previous cribs so as to sustain this load. It was calculated that the maximum horizontal force would be of the order of 20 tons.

When the bottom section of a crib had been wedged to the positioning frame, and was all but completely immersed, buoyancy having released the traveller of most of the vertical load, the next section would be placed on top and strapped to it. In this way, the joining of sections below water level was avoided, and a crib, weighing say 120 tons, could be placed quite quickly in position.

Once placed, the cribs were filled with rock, the wedges pulled, and the positioning frame released and raised to the surface. Two reinforced-concrete bearing pads were constructed on each filled crib to carry the beams spanning from crib to crib. The railway tracks were then extended so that the traveller could be moved forward 32 feet in readiness to repeat the series of operations. Once experience was gained the complete cycle of operations regularly took only 8 to 10 days.

The process was repeated until crib No. 8 had been placed, after which the traveller was moved back temporarily to permit construction of the cross-wall to proceed, following which it moved forward to complete cribs Nos 9–16 for cofferdam No. 3.

Owing to the heavy current it was found necessary to construct a second sheet-pile wall, 9 feet upstream of the main line of piles, to protect the divers while engaged on sealing operations.

Originally it had been intended that divers should excavate any loose material overlying the rock and form a seal by means of a triangle of clay. Unfortunately, it was found that for a length of about 100 feet no rock could be proved, and a test boring at the south-east corner showed a depth of 26 feet of broken material. Moreover, the divers discovered a number of seams running in a north-south direction. To seal this area it was therefore decided to form a piled box, 20 feet wide, which was filled with about 6 feet of murram. A 60-foot-long grout curtain was formed to cut off any water which might penetrate below the murram blanket.

The downstream arm and the cross-wall were formed by smaller cribs (17 feet by 15 feet in plan) placed by two cranes manufactured on site, having two fixed jibs each and capable of lifting 25 tons. The crib to be placed was assembled on the one which had just been placed. After it had been lifted the crane, with the crib suspended, was moved forward until it was able to lower the crib into its correct position. They were held in position whilst being lowered by wire ropes attached to the upstream arm of the cofferdam, one leading downstream and two holding the crib tight to vertical guides attached to the previously placed crib. These small cribs were timber-sheathed on the outside, and provided with cantilevered pile guides so that, after placing, sheet-piles could be driven 7 feet outside the timber sheathing, providing a corridor which was sealed with murram after the river bed had been cleaned by divers.

Owing to the force of the water and the irregular river bed, two of the downstream cribs moved about 2 feet in spite of support from cables taken to the upstream cribs. This, however, did not seriously affect the programme, and the sheet piles were re-aligned to suit the new position of the cribs.

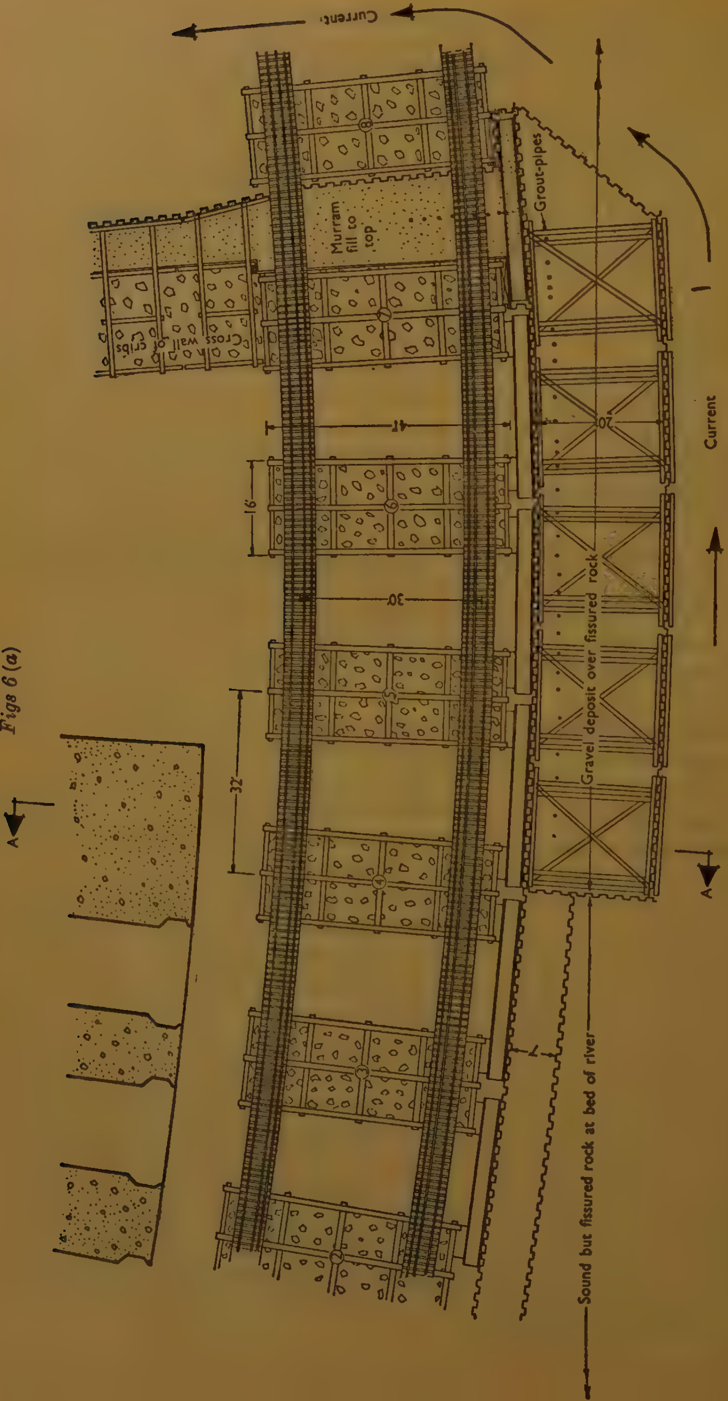
Cofferdam No. 3.—The cribs for the upstream wall of cofferdam No. 3 were placed whilst the dam was under construction in cofferdams Nos 1 and 2. As soon as these were in position a travelling 6-ton derrick was erected on them, which was used for placing the walings and sheet-piles. The timing was such that all preparatory work for placing the piles was finished at the same time as the river was diverted through the gaps left on the western side of the main dam. The complete diversion was accomplished gradually by the pile placing, which had to be done under a head of water ranging from 5 to 8 feet. In order to guide each pile over the bottom waling, while being tapped home by a McKiernan-Terry automatic hammer, a wire rope was taken from the toe of the pile upstream to a return block and back to a winch on the cribs.

It was found necessary to extend the 20-foot box of cofferdam No. 2, and after divers had cleaned the area along the toe of the sheeting, clay and clay bags were placed to form a seal, and the grout curtain was extended westward. At the eastern end, a murram bank was tipped out from the shore.

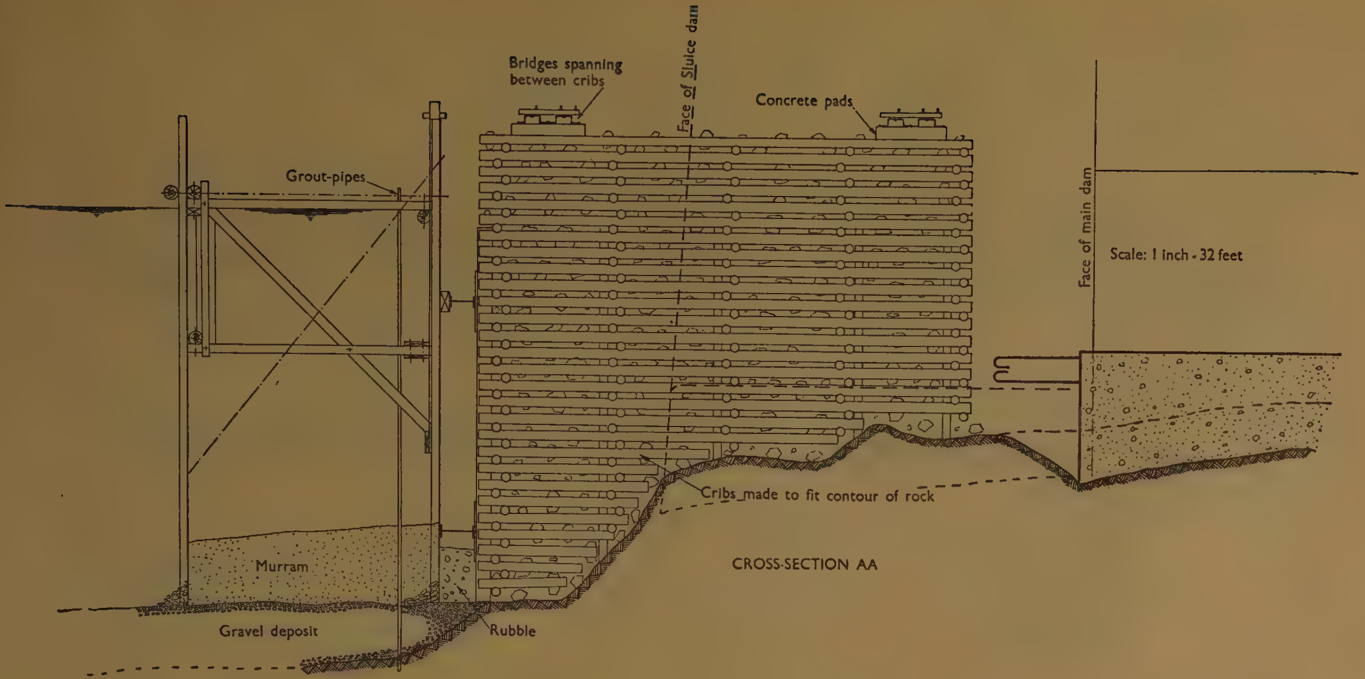
The downstream arm, where the water was shallow, was formed by a murram bank with a rubble filter, a few piles being driven into the bank to make good to the downstream piling of cofferdam No. 2.

It took about 5 weeks to dewater this cofferdam, owing to an extensive leak through jointed rock at the south-east corner. This was staunched by a combination of grouting and tipping of a further murram blanket upstream of the cofferdam. The pumping necessary to hold the water was about 500,000 gallons per hour.

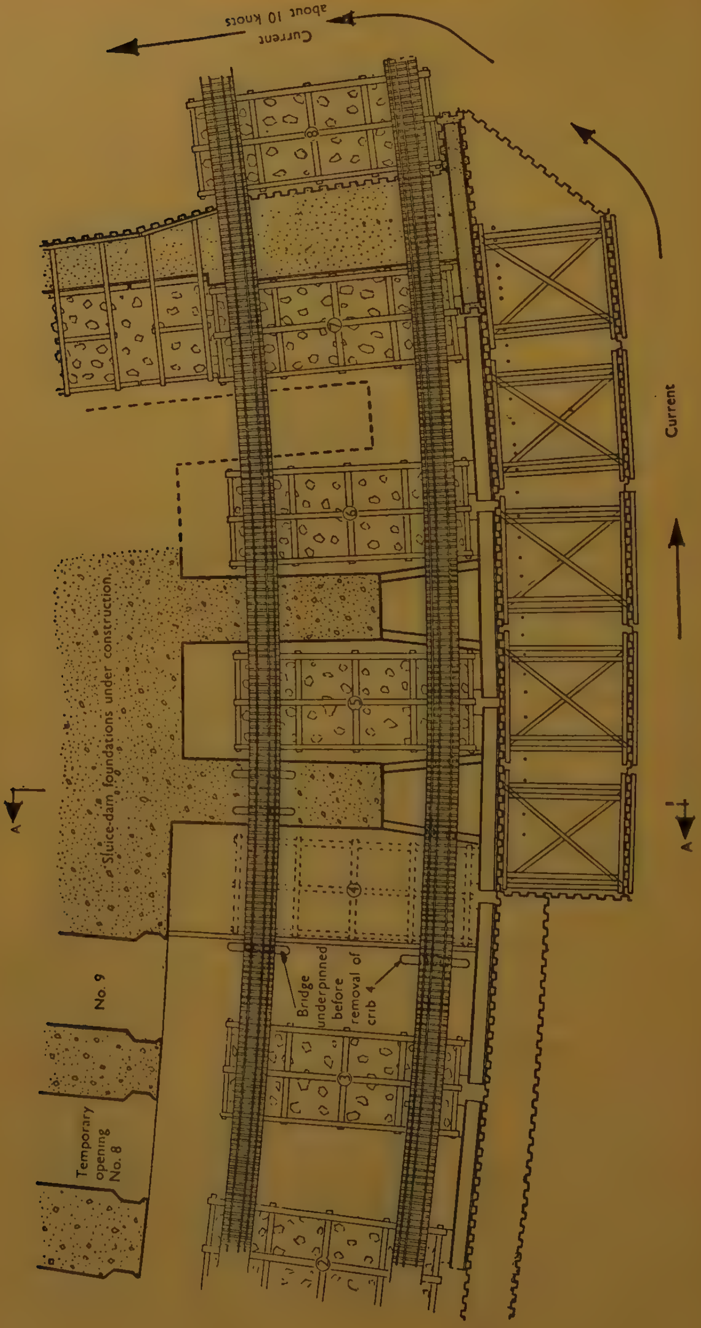
Underpinning upstream of the sluice dam (*Figs 6*).—The main dam is founded on an amphibolite ridge which runs across the river and falls away very sharply on the upstream side. It was therefore a great advantage to keep the upstream piles as close as possible to the dam, but special measures were necessary over the area where the sluice dam projects out in front of the upstream face of the dam. The problem was overcome by removing the cribs in front of the sluices after alternative supports for the piles had been provided by constructing, between the cribs, concrete buttresses reinforced with rails and designed to transmit the load from the walings to the dam.


Current
Grout-pipes
Murram fill to top
Cross wall of cribs
8
7
6
5
4
3
2
47'
16'
30'
32'
20'
7'
Current
Gravel deposit over fissured rock
Sound but fissured rock at bed of river
A
A

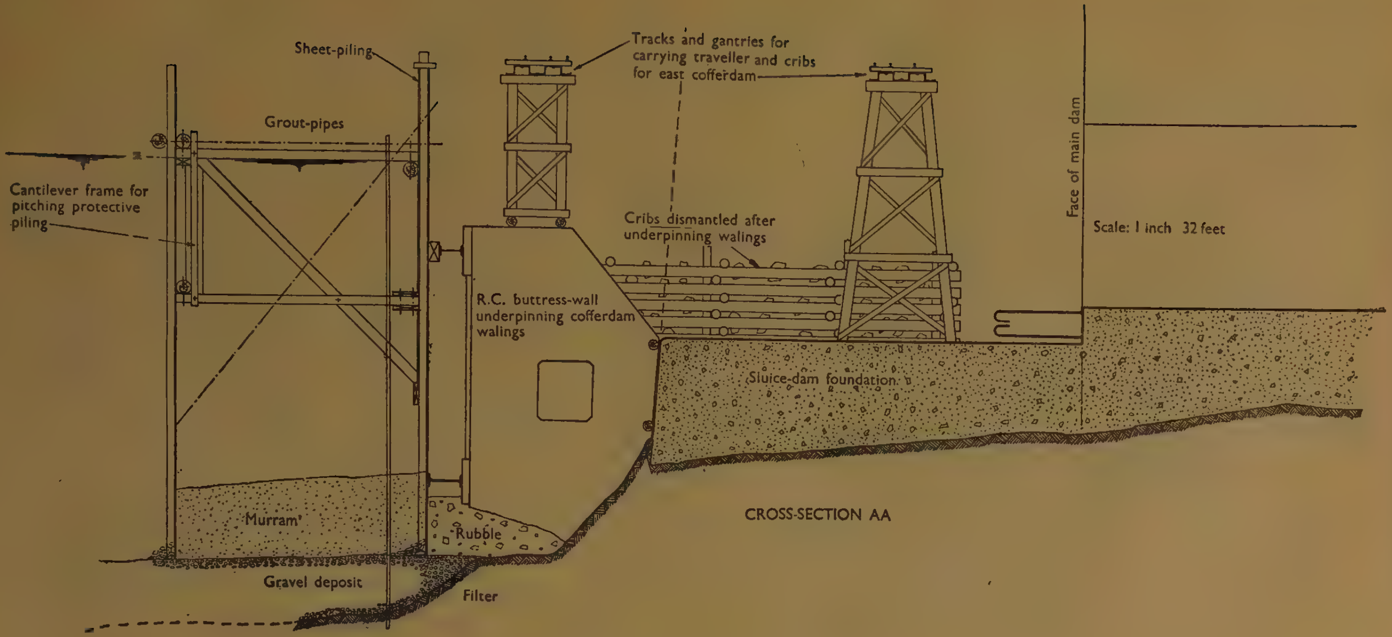
Figs 6 (a)



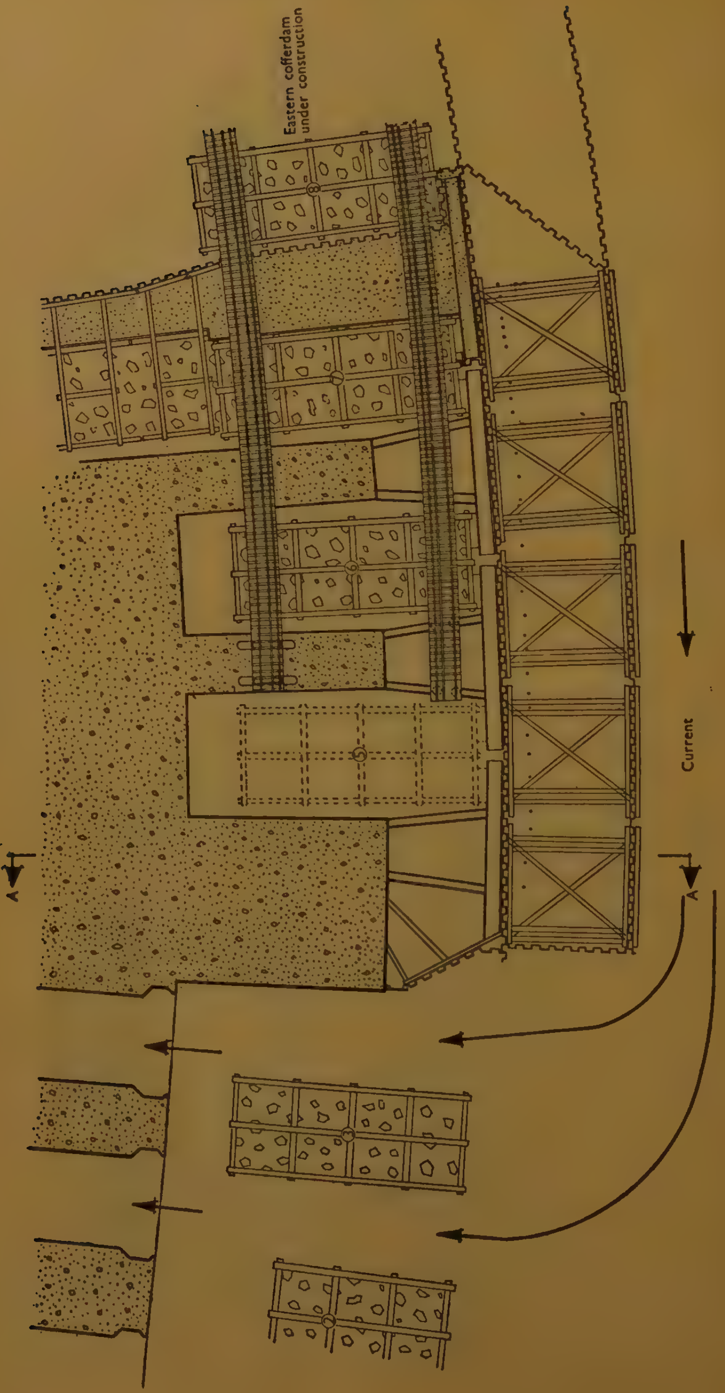
STAGE 1 OF UNDERPINNING AT SLUICE DAM


Current
about 10 knots
Current
A
A
Sluice-dam foundations under construction
No. 9
Temporary opening No. 8
Bridge underpinned before removal of crib 4
8
7
6
5
4
3
2

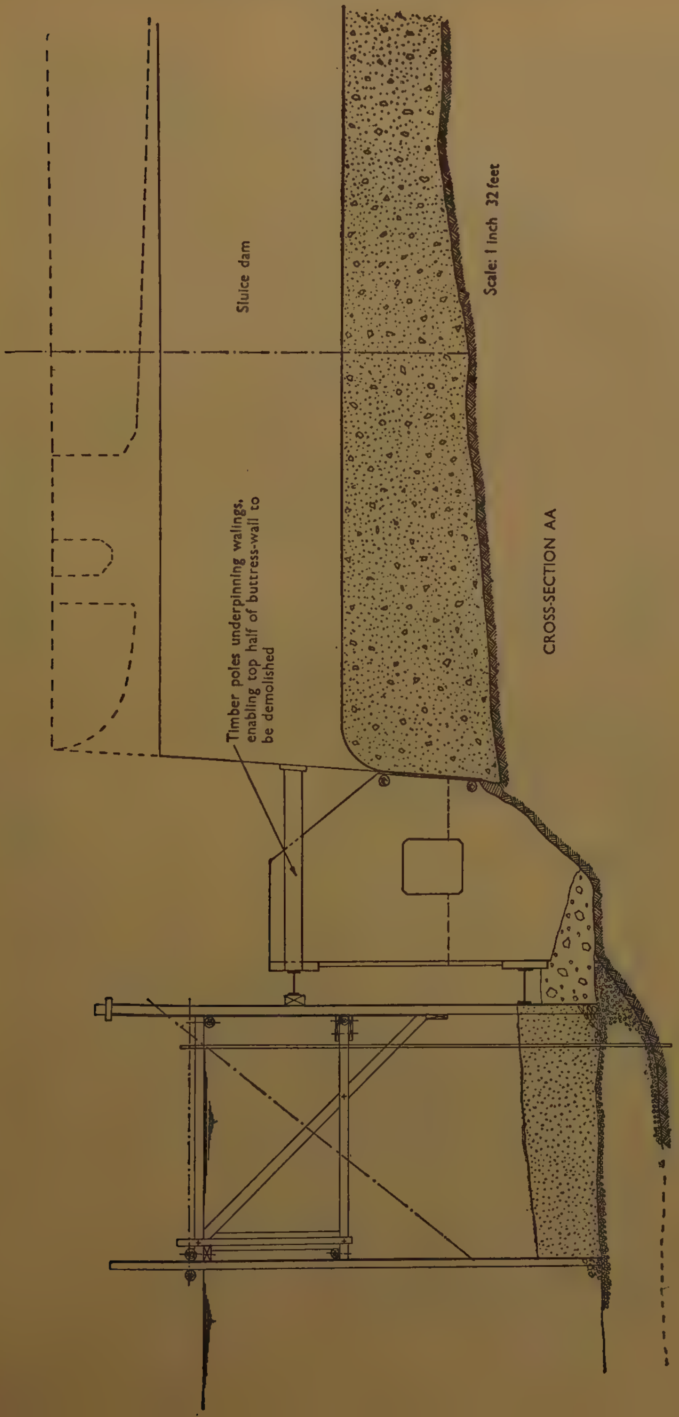
Figs 6 (b)


Sheet-piling
Tracks and gantries for carrying traveller and cribs for east cofferdam
Face of main dam
Grout-pipes
Cantilever frame for pitching protective piling
Cribs dismantled after underpinning walings
Scale: 1 inch 32 feet
R.C. buttress-wall underpinning cofferdam walings
Sluice-dam foundation
CROSS-SECTION AA
'Murram'
Rubble
Gravel deposit
Filter

Stage 2 of Underpinning at Sluice Dam

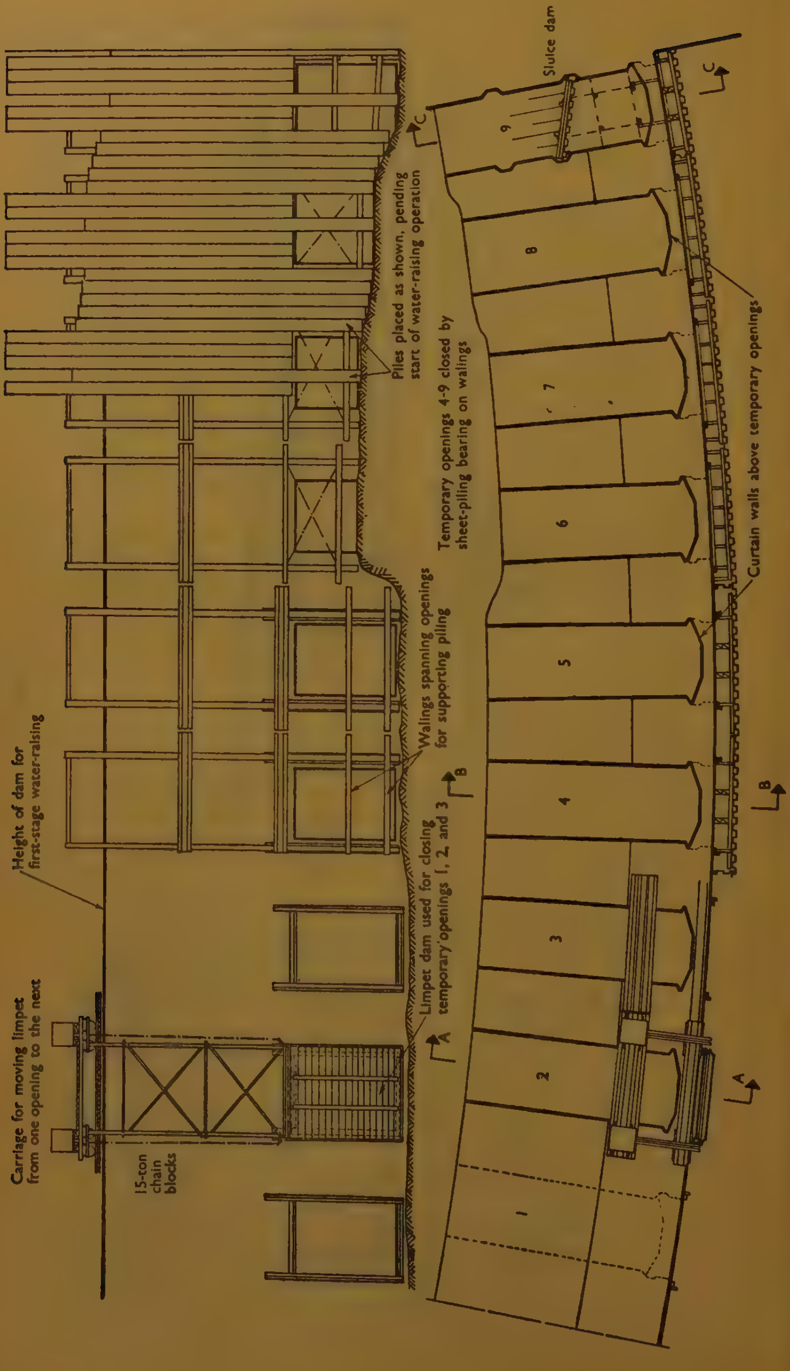


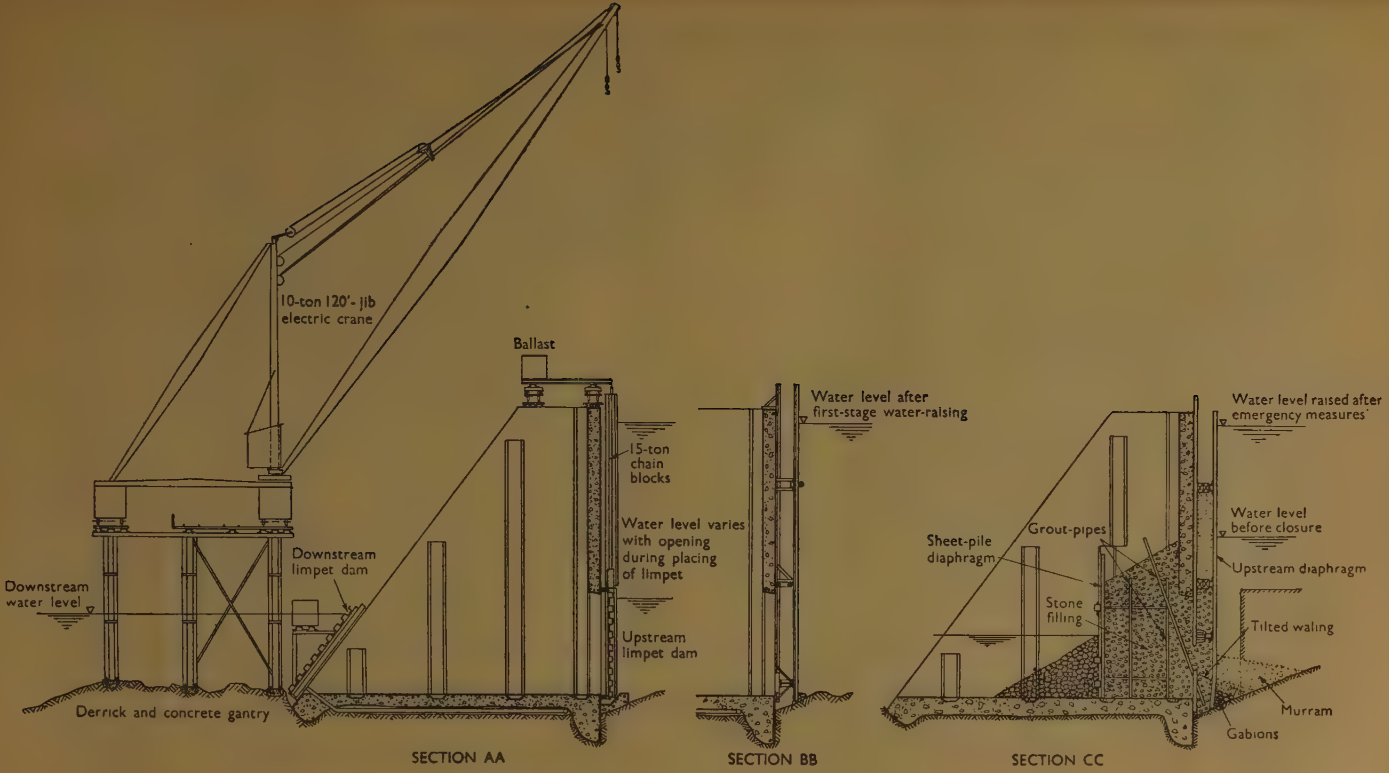
Figs 6 (c)


Sluice dam
Scale: 1 inch 32 feet
Timber poles underpinning walings, enabling top half of buttress-wall to be demolished
CROSS-SECTION AA

Stage 3 of Underpinning at Sluice Dam

Figs 7


Carriage for moving limpet from one opening to the next
15-ton chain blocks
Height of dam for first-stage water-raising
Limpet dam used for closing temporary openings 1, 2, and 3
Walings spanning openings for supporting piling
Temporary openings 4-9 closed by sheet-piling bearing on walings
Piles placed as shown, pending start of water-raising operation
Curtain walls above temporary openings
Sluice dam
A
A
B
B
C
C
1
2
3
4
5
6
7
8
9


10-ton 120'- jib electric crane
Ballast
15-ton chain blocks
Water level varies with opening during placing of limpet
Upstream limpet dam
Downstream limpet dam
Downstream water level
Derrick and concrete gantry
SECTION AA
Water level after first-stage water-raising
SECTION BB
Water level raised after emergency measures
Water level before closure
Upstream diaphragm
Tilted waling
Murram
Gabions
Grout-pipes
Sheet-pile diaphragm
Stone filling
SECTION CC

Method of Closing Temporary Openings

Later, when the whole of the front face of the sluice dam was sufficiently high, the walings were re-strutted to the concrete using round poles, after which the concrete buttresses were in their turn removed, and the space between dam and piles was cleared down to a level 10 feet below sluice-invert level.

In order to permit first-stage water-raising before sluices Nos 5 and 6 were ready, a temporary sheet-pile cofferdam, attached to the foundation concrete, was erected round these intakes and strutted from the downstream portion of the dam, which, since it did not contain any gate guides, was much simpler to construct and permitted the guides and grouting to be done after water-raising.

The removal of the piling, walings, and struts upstream of the sluice dam was done by flooding between dam and piling, with gates in sluices Nos 1–4 closed.

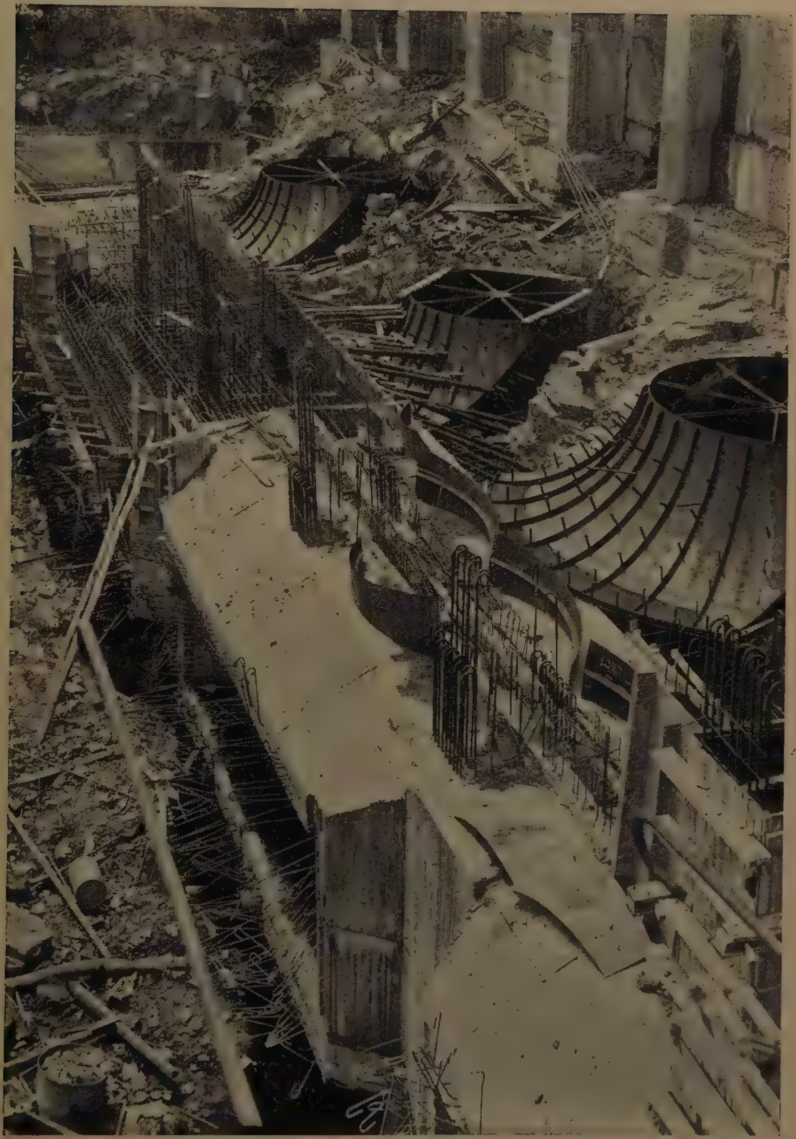
Blocking the temporary openings.—Since the first three openings could be blocked without raising the water above about + 3,670, it was decided to use a limpet dam (*Figs 7*) to close these openings one after the other. This limpet was constructed using sheet-piles with timber bearers and was lowered into position, sliding on previously prepared fixed ways. In order to overcome the friction during the last few feet of lowering, the limpet was ballasted and hammered down with a pile hammer. After placing this upstream limpet a similar downstream limpet was placed and ballasted with about 30 tons, following which the opening was dewatered and concreted.

A different system had to be adopted for openings Nos 4 to 9 in order to save time and because of the greater head involved. Sets of walings were placed across the openings and sheet-piles driven in front of them. In order to delay water-raising as long as possible, only the piles in front of the concrete piers and one double pile in the centre of each opening were driven down to rock in the first instance. Even so, the effect of the pressure of water was such that when it came to closure, a 4-foot taper was necessary, this in spite of stiffening plates having been welded into the bosoms of the piles.

Before completing the entire sheet-pile curtain-wall it was necessary to restrict flow of water at Ripon Falls to the maximum possible extent, and to bring sluices Nos 1–4 into commission. It was found necessary again to use winches to pull the toes of the piles, which had been shaped into wedges, over the bottom walings.

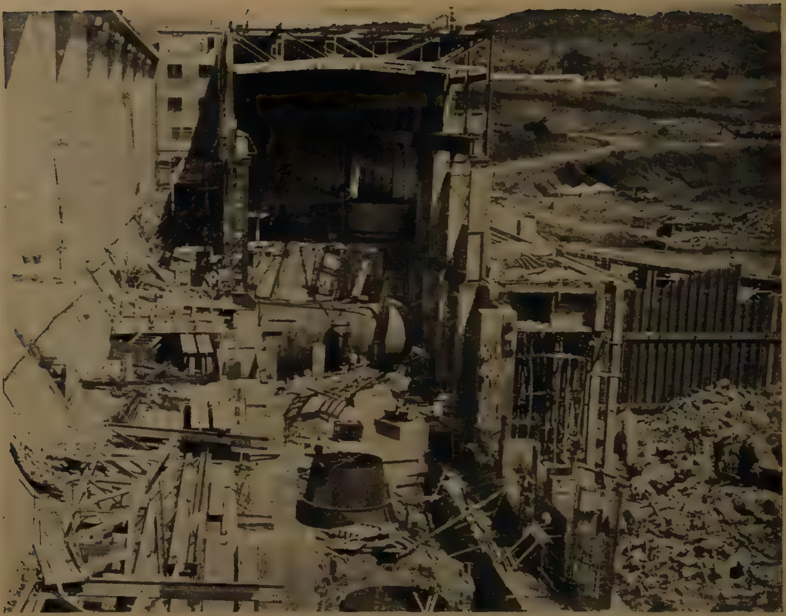
Great difficulty was experienced in closing opening No. 9, owing to the fact that a 12-inch-by-12-inch R.S.J., 50 feet long, which had previously been used as a strut alongside the sluice dam, had dropped to the river-bed during dismantling, and had ended up across the bottom waling of this opening where it interfered with the completion of pile driving. At this stage it was impossible to extract the piles in question and it was therefore necessary to remove the joist by a diver, working next to sluice No. 1

Fig. 16



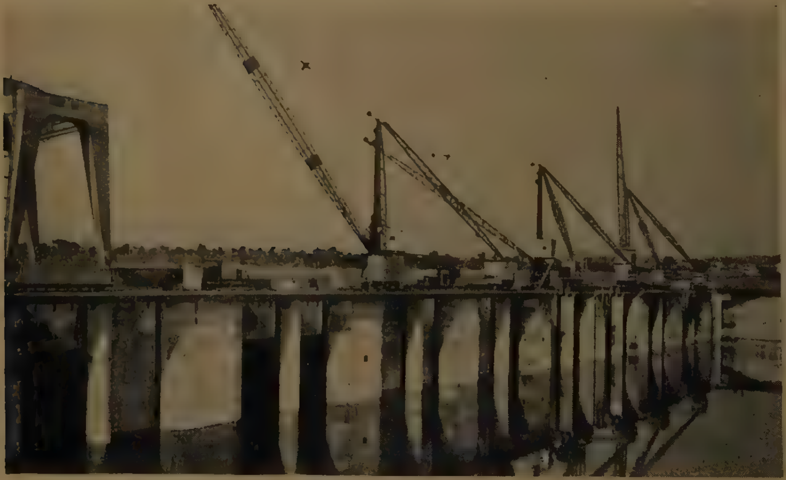
Power-House Foundations. Shuttering for the Beam over the Draft Tube

Fig. 17



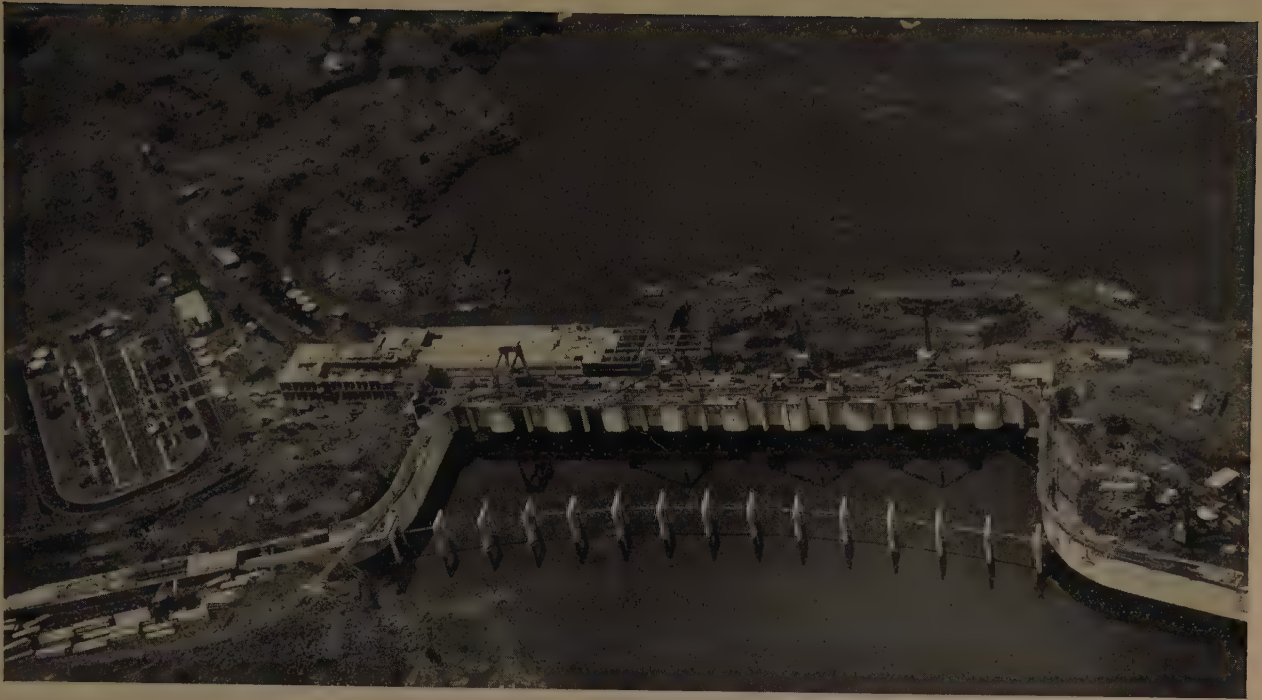
Construction of Power House. Intake Dam is on the Left and Diaphragm on the Right

Fig. 18



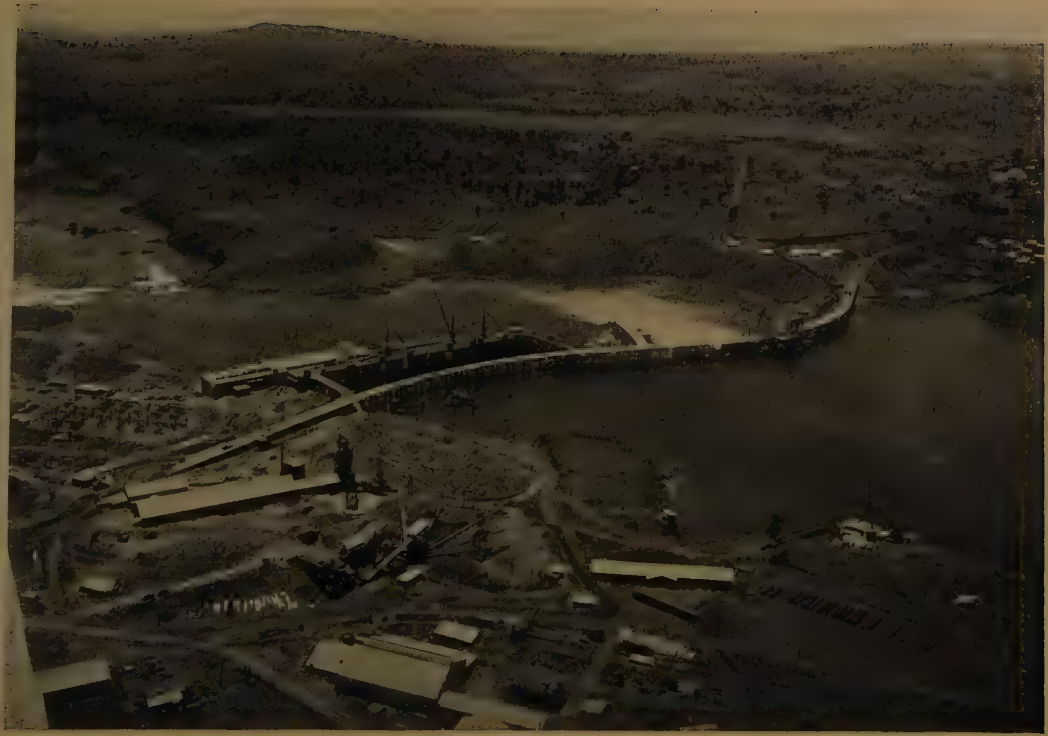
Upstream Face of Intake Dam. The Cranes are being used for Erection of the Power House

Fig. 19



View of Head-race and Intake Dam before Construction of Road-Bridge Deck. The Structure on the Left-Hand End of the Intake Dam is the 30-ton Crane used for Handling the Gates

Fig. 20



VIEW FROM WEST BANK LOOKING DOWNSTREAM. JINJA AIRSTRIP IS IN THE BACKGROUND AND WORKSHOPS IN THE FOREGROUND (closed during the operation), attaching slings from two derricks to its upstream end. In the act of wrenching it free some pile toes were damaged and the waling tilted so that it became practically impossible to complete a reliable curtain. To overcome these problems for sealing the opening a waling grillage was lowered into two recesses left in the concrete walls as key-ways, about 16 feet downstream of the dam face. Against this framework, 30-foot-long piles were placed and the space between this diaphragm and the curtain wall was filled with rubble and 3-inch stone so as to transfer the hydrostatic pressure from the curtain to the diaphragm. Four-inch-diameter pipes with welded bracings were placed within the stone to permit this portion of the dam to be "Colcreted" after placing the concrete downstream of the diaphragm. After water-raising the considerable leakage under the piles was sealed by placing gabions (3-inch stone bound up in wire netting) and rubble, followed by stone graded gradually to smaller sizes, and finally murram.

A downstream dike of rubble and murram enabled the temporary openings to be dewatered and concreted.

Power-house cofferdam.—It was found at an early stage that there was a gravel deposit immediately downstream of the power house, on the site of the cofferdam within which the power house was to be built. It was therefore decided to move the intake dam and power house 90 feet southward, thereby bringing the cofferdam on to a much better foundation. This entailed an increase in the rock excavation, but decreased the volume of concrete in the head-race dams. Cofferdam construction was made easier and all that was needed was a shallow murram bank with an inside filter toe and some rubble protection on the water side.

In order to enable the first three sets to be commissioned ahead of the complete construction of ten sets, a temporary diaphragm was constructed between sets Nos 3 and 4, consisting of two lines of Krupp III piling. These were tied together with tie-rods and the space between filled with gravel.

SITE LAY-OUT

The whole of the static installations (see Fig. 1, Plate 1), such as the workshops and central aggregate and concrete plants, were sited on the west bank above the top water level after flooding. Metre-gauge track was adopted for it enabling direct connexion to be made to the E.A.R. & H. main line.

Ten-ton derricks with 120-foot jibs were the standard lifting appliance used for shutters, concrete, etc. Twelve of these were employed, and were repeatedly moved from one position to another; in four places they were mounted on bogies and traversed: in the head-race for constructing the intake dam; in the tail-race for the power house; on top of the intake dam for the power-house superstructure, cable ducts, and top work of the intake dam; and downstream of the portion of the main dam west of the sluice dam and on the sluice dam itself. Here a structural steel gantry was constructed using sheet-piles and steel sections, which carried a double railway track in addition to two derricks mounted on bogies and gabbards. It was built inside cofferdams Nos 1 and 2 and was designed to withstand the force of the water passing through the gaps.

Another gantry was built from the eastern bank to join cofferdam No. 2 prior to the construction of cofferdam No. 3, designed to carry a double track across to the east bank to enable the eastern haunch of the dam to be built before the completion of cofferdam No. 3. It was built with eucalyptus poles placed in position using guide frames cantilevered out from the completed work. A cable was taken from every bent of the gantry to an anchorage upstream, and the structure was further stabilized by 1-ton concrete blocks being attached to the upstream bottom walings.

Two suspension bridges, of 325 feet and 156 feet span respectively, provided temporary access across the river and enabled European employees to cross from the east bank where they were housed.

EXCAVATION

About 285,000 cubic yards of amphibolite had to be excavated, of which about 100,000 cubic yards were crushed for use as aggregate. The balance of the rock required for crushing was obtained from quarries opened up on the site, one on an island at the Owen Falls and the other on the west bank opposite this island.

All good rock was either crushed and used as aggregate, placed in the form of cyclopean blocks, or used for filling the cofferdam cribs. Cyclopean blocks were used in the early stages, mainly to save cement, but in the particular circumstances progress was found to be affected, and when local cement became available their use was abandoned.

Either wagon drills with 2⅜-inch tungsten-carbide detachable bits or jackhammers with 1¾-inch bits were used for drilling; 85 to 95 cubic yards were excavated per bit.

Blasting was carried out with 8-inch-by-1½-inch 60-per-cent ammon gelignite, the overall average consumption being 0·75 lb. per cubic yard. For open work, such as in the quarry which was opened on an island in the river, 0·4 lb. per cubic yard proved sufficient for primary blasting.

Generally 1½-cubic-yard excavators were used for excavation, usually as face shovels, though back-actor equipment was found most useful, particularly in mucking out the bottoms of foundations. A fleet of twenty 4½-cubic-yard Diesel dumpers ran the rock either to stock heap or direct to the crushing plant.

CONCRETE PRODUCTION

A single 25-inch primary cone crusher fed two 16-inch secondary cone crushers by vibrating feeders. The rock then passed over grizzlies and was taken by belt conveyor to the screening plant (see Figs 8, Plate 2) which consisted of one triple- and one double-deck vibrating screen. Rejects passed to a Gyrasphere crusher or to a 20-ton balancing hopper which fed two hammer-mills. The product was passed back to the screening plant. A dust-extraction plant enabled a proportion of the fines to be extracted at the tertiary crushers and screens. The coarse aggregate, obtained from foundation excavations (mainly from the power-house site) or from quarries opened on the site, was crushed to the following sizes :—

$0\text{–}\frac{3}{16}$ inch ; $\frac{3}{8}$ inch ; $\frac{3}{4}$ inch ; $1\frac{1}{2}$ inch ; and 3 inch.

A ratio of one-third crusher fines to two-thirds natural sand was used ; the latter was transported by water to Jinja jetty from a lake shore deposit about 15 miles away. Two tugs and eight steel barges were used, the sand being grabbed from barge at Jinja and transported by lorry to the site, where it was washed by being pumped into a tank with a balanced bottom gate, which opened when the tank was full, releasing washed sand to stockpile, the dirt being washed away with the water flowing over the tank top.

More than half the cement came from Europe in paper bags, storage having been provided for 6,000 tons. The remainder came from the Tororo factory of the Uganda Cement Industry Ltd, which started operating early in 1953. This cement, which was a low-heat type, was handled in bulk by two hundred and fifty $2\frac{1}{2}$-ton containers, feeding a 600-ton silo built adjacent to the central mixing plant to act as a balancing tank.

Automatic weigh-batching was adopted, the mixing being done by two tilting-drum mixers, pneumatically tipping into two 4-cubic-yard wet hoppers situated between them. The wet hoppers were arranged to discharge above either of the two tracks which ran beneath them. The maximum output in 24 hours was 1,100 cubic yards.

The products of the screening plant were conveyed to stock heaps by inclined belts. Reclamation from stock heap was by belt conveyors running in tunnels below the stock, and thence to the central batching plant, also by belt. The tunnels below the stockpiles were constructed using eucalyptus poles. Cement was fed to the plant by archimedian screws and vertical bucket-conveyors.

CONTROL OF MATERIALS

A site laboratory, equipped early in the contract, was used continuously for testing permanent materials. By courtesy of the Uganda Electricity

Board, assistance with regard to testing of materials was often given to other less favourably placed local authorities.

The total number of briquettes tested exceeded 1,500, and the total number of cubes crushed 4,000. When a low-heat type of cement to B.S. 1,370 became available from Tororo during 1953, it was necessary to acquire additional testing equipment. Brief descriptions of the different classes of concrete and typical comparisons between performances of imported and local cements are given in Appendix III.

CONCRETING

Bottom-dump skips of 2 cubic yards capacity, carried on bogies and hauled by 35–40-horse-power Diesel locomotives, were used for conveying concrete from the central batching plant to the work. In general, derricks were used for handling the skips, although 38RB. excavators were employed at places not covered by the derricks. Maximum rate of placing was about twenty skips per hour per derrick. Vibrators were used almost universally.

GROUTING

A grout curtain cut-off was provided under the main, intake, and head-race dams, to seal fissures in foundation rock, the quality of which was found to vary considerably over short distances. In all, 20,345 lineal feet of rock drilling was carried out and 98 tons of imported cement injected, the maximum cement intake recorded for one hole having been 4·1 tons.

Holes with a rake of 1 to 5 were drilled at 10-foot centres through the upstream face of the dams to meet rock approximately in the middle of the excavated cut-off trench, alternate holes being treated as primaries, the others as secondaries. Generally the former were grouted to a maximum depth of 60 feet, and the latter to a maximum depth of 30 feet, according to the following specified drilling stages :—

1st stage	2nd stage	3rd stage	4th stage
0–14 feet	14–30 feet	30–50 feet	50–60 feet

In cases where really bad rock was encountered, drilling was carried out in 10-foot stages and each stage was regrouted until satisfactory.

First-stage drilling was carried out with jackhammers, using 1⅝-inch diameter cruciform tungsten-carbide bits ; subsequent stages were drilled using 42-millimetre diamond bits.

Grouting was allowed to proceed after 15 feet of concrete had been deposited over the base pour in contact with foundations according to the following general scale of pressures.

Depth: feet	Primary: lb. per sq. in.	Secondary: lb. per sq. in.
0–10	50	65
10–20	65	85
20–30	80	110
30–50	100	130
50–70	100	150
70–90	120	—

Before grouting, holes were usually flushed by air and water, and after grouting holes were flushed and allowed to stand for at least 24 hours before applying a water test. Pressures for water testing were 10 lb. per square inch higher than those specified above and for intermediate stages a drop of 30 inches was accepted, but for the final test a drop of not more than 12 inches over a period of 10 minutes was specified. Holes which failed the water test were required to be regrouted at stage pressures which were 10 lb. per square inch higher than those specified above. The mix ratio generally commenced with 10 : 1 by volume, water-cement, thickening in isolated cases to as much as 1 : 2 if the grout was taken without stage pressures increasing above those specified. For the east cut-off, where the material grouted consisted of decomposed amphibolite, the starting ratio was 600 : 1, gradually thickening to 20 : 1. Great difficulty was experienced in grouting this series because holes continued to "fall in" as soon as drill-rods were withdrawn, and the close texture of the material would not easily accept grout.

SHUTTERING

For plain faces, steel cantilevered shutters, fabricated on site were used, held in position by coarse-threaded bolts, or by bolt and embedded nut. Purpose-designed steel shutters were also used in a number of places, such as for the upstream nosing of the splitter walls in the intake dam, and steel was used whenever it was justified by the number of uses (about fifty). Treated plywood was used for the main-dam sluices to get a really good finish and to conform to the intake curves. It was also employed in the power house and for precast work.

Elsewhere, local timbers, mainly Mpewere (*Piptadenia buchananii*), Podo (*Podocarpus spp.*) and Musisi (*Maesopsis eminii*) were used for both shuttering and for general construction. It was, however, only possible to get from one to three uses out of such shutters. Experiments were tried with Muhimbi (*Cynometra alexandri*), but it is very hard wood, difficult to work, and expensive ; its greater durability did not fully make up for these disadvantages.

PRECAST WORK

Precast concrete was employed wherever possible, notably for the switching-station structures, the power-house cladding, the road above the dam and the bridge across the head-race.

The power-house cladding elements were cast with a rough pink granite finish with artificial joints, and were placed in position with chain blocks. The roof elements were cast on the intake dam and placed in position by derricks.

The members making up the road had a maximum weight of 15 tons, and were, for the most part, handled by a derrick erected on three of the barges that had previously been used for supplying sand. The west-dam roadway was constructed prior to flooding by means of a 15-ton caterpillar crane.

BANK PROTECTION WORKS

A survey of the river banks between the dam site and Ripon Falls including examination of core-drilled samples, revealed that in two major places, where the main railway line is located close to the edge of the banks, rockfill would be necessary as protection works and to stabilize the banks after water-raising. On the east bank, on both sides of the east-approach embankment, about 27,500 cubic yards of consolidated rockfill were placed in layers. On the west bank the problem was overcome by the East African Railways and Harbours agreeing to divert the track about 200 feet westwards for a distance of approximately one mile, thus avoiding approximately 25,000 cubic yards of rockfill at a place where access for heavy plant was difficult.

WATER-RAISING

Both the tail-race and the head-race were flooded before raising the water level, the former by removing the pumps from the cofferdam, and the latter by pumping in water. At that time two of the four completed sluices were provided with the permanent control gates on power operation ; in the other two, the gates were suspended in the gate wells and could only be lowered in an emergency. Also, the sheet-piles covering the temporary openings had not yet been completely sealed.

Filling of the reservoir was effected mainly by increasing the flow at Ripon Falls, but acceleration was achieved by the periodic closing of one sluice gate. Provided there was no change in the level of Lake Victoria, ponding level could be altered simply by adjusting the lack of balance between the inflow and outflow. Water-raising proceeded at an average rate of 4 feet per hour, up to first stage level of + 3,697 U.S.D., which allowed the first turbine to be operated on reduced load. Final raising was dependent on the rate of completion of the upper portions of the dam and of protection works to river banks and to the hinges at Jinja Bridge. At first-stage filling-level, the head on the sluice invert was about 49 feet, and each sluice discharged about 5,600 cusecs.

WORKSHOPS AND SERVICES

In view of the isolation of the site, and the difficulty of obtaining spares or engineering services quickly, it was evident that the works had to be more or less self-supporting. An adequate organization was therefore set up, with separate machine, blacksmiths, drill, electricians, welding, and wood-working shops. (See Appendix II.)

Temporary electricity was supplied by the Uganda Electricity Board from a ring-main service which fed five transformers on site, from which the contractors tapped at 440 volts. The maximum demand was 2,500 kVA. and the total number of units consumed during the peak year, 1953, amounted to 2·7 million kWh. Most of the pumps for dewatering cofferdams were electrically driven and few interruptions of power supply occurred during the 2-year period while heavy pumping was in progress. Compressed-air mains were provided throughout the site, portable compressors being used to augment the supply where necessary.

ACKNOWLEDGEMENTS

The Authors would like to thank the Uganda Electricity Board, the Consulting Engineers (Messrs Kennedy & Donkin and Sir Alexander Gibb & Partners), and the Owen Falls Construction Company, for permission to present this Paper.

During the early stages of the contract progress on the scheme was facilitated considerably through the valuable assistance rendered by the Public Works Department and the Geological Survey Department of Uganda, in connexion with early surveys and the interpretation of borehole cores respectively. Throughout the contract, assistance was also rendered by the District Commissioners, Labour, Forestry, Medical and other Uganda Government Departments, and the Authors gratefully acknowledge their indebtedness to all these Authorities.

Acknowledgement is also due to Mr Bijl of the Owen Falls Construction Company, and to Mr M. W. Baldwin, B.Sc., A.M.I.C.E., of Sir Alexander Gibb & Partners (Africa), who assisted materially in the preparation of this Paper.

Although the Uganda Electricity Board have approved of the presentation of the Paper, the views and estimates expressed therein are the personal responsibility of the Authors and are not necessarily those of the Board.

The Paper is accompanied by twelve photographs and nine sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared, and by the following three Appendices.

APPENDIX I

List of Principal Contractors and Sub-Contractors

(1) *Civil engineering works*
Main contractor :—The Owen Falls Construction Company formed by the following firms :—
Christiani & Nielsen Limited.
Dorman Long & Co., Ltd.
Edmund Nuttall Sons & Co. (London) Ltd.
Hollandsche Beton Maatschappij N.V.
Internationale Gewapendbeton Bouw N.V.
J. L. Kier & Co. (London) Ltd.
Nederlandsche Anneming Maatschappij N.V.
Nederlandsche Beton Maatschappij Nato N.V.
Principal sub-contractors :—
Electrical installations :—Electrical Installations Ltd.
Heating and ventilation, and hot- and cold-water services :—Norris Warming Co. Ltd.
Lifts :—Marryat & Scott, Ltd.
Furnishings :—Waring & Gillow, Ltd.

(2) *Structural steelwork*
Main contractor :—Redpath Brown & Co. Ltd.

(3) *Intake gates, draft-tube gates, flood control gates and associated equipment*
Main contractor :—Glenfield & Kennedy Ltd.
Principal sub-contractors :—
Cranes :—
Sir Wm. Arrol & Co. Ltd.
Motherwell Bridge & Engineering Co., Ltd.

(4) *Erection of gates*
Main contractor :—Blane & Co., Ltd.

APPENDIX II

List of Construction Plant

Earth-moving plant

4	1½-c.y. excavators with crowd shovel equipment and two sets of dragline equipment.
1	backactor equipment for excavators.
2	¾-c.y. excavators with crowd shovel equipment.
2	dragline equipments for excavators.
2	D-8 bulldozers.
1	D-6 bulldozer.
3	Scrapers: 7, 9, and 13 c.y.
17	4½-c.y. Diesel dumpers.

Cranes and derricks

9	10-ton 120-ft-jib electric derricks.
1	7-ton 100-ft-jib electric derrick.
1	5-ton 120-ft-jib electric derrick.
1	15-ton 60-ft-jib Diesel-electric caterpillar crane.
1	6-ton 12-ft-gauge locomotive crane.
2	5-ton 4′ 8½″-gauge locomotive cranes.
1	3-ton mobile hydraulic crane.
1	20-ton caterpillar-tractor crane.

Grabs and crane equipment

1	45/36-c.ft and two 27/22-c.ft general purpose grabs.
2	34/27-c.ft sand grabs and one 2-c.y. rock grab.
	Miscellaneous tongs and rock trays.

Crushing and screening plant

1 25-B and 2 16-B cone crushers.
1 48-inch "Gyrasphere" crusher.
2 F-type hammer mills.
1 3-deck and one 2-deck vibrating screens.
7 vibrating feeders.
1 dust extraction plant, conveyers, chutes, hoppers, etc.

Batching and concrete-mixing and placing plant

1 central batching plant, with six aggregate and one cement compartments, automatic weigh batching, two 2-c.y. tilting-drum mixers and two wet hoppers; two cement feeds: one from the cement-store (bagged cement) and one from the 600-ton two-compartment bulk-handling silo, which was fed by a 5-ton overhead crane which lifted 2½-ton cylindrical cement-containers direct from permanent-way rolling-stock.
6 Diesel and petrol portable concrete-mixers ranging from 7/5 to 14/10 capacities.
20 2-c.y. bottom-dump skips.
20 pneumatic and seven electric vibrators.

Sand-washing plant

1 sand washing plant, consisting of a 5-inch centrifugal pump, pumping into a boiling box and cone unit.

Rolling stock

6 40-h.p. and eight 48-h.p. Diesel locomotives (metre gauge).
40 flat-topped bogies, 24 jubilee waggons, and heavy duty bogies for carrying cribs, derricks, etc.

Pneumatic plant and equipment

4 500-c.ft stationary electric compressors.
2 500-c.ft and two 260-c.ft portable Diesel compressors.
8 drifter drills with wagon mountings.
6 rock drills (CP32).
4 rock drills (CP225).
12 rock drills (CP42).
6 concrete breakers (CP117).
Miscellaneous wood borers, dropping hammers, riveters, and other light pneumatic tools.

Pumps

1 10-inch and eight 8-inch centrifugal electric pumps.
2 6-inch centrifugal vertical-spindle pumps.
1 9-inch and one 7-inch Pulsometer steam pumps.
Miscellaneous sludge, water-supply, and other pumps.
A total of more than 50 pumps were used.

Piling equipment

1 boiler (12′ 10″ high by 5′ 5″ dia.).
1 No. 7 McKiernan-Terry automatic hammer with extracting equipment.

Grouting equipment

3 diamond drills (CP.55).
1 grout pan.
3 cementation pumps.
1 Colcrete mixer.

Drill-sharpening shop

2 drill sharpeners.
1 oil furnace.

Winches and hoists

35 crab winches ranging from 2 to 10 tons capacity.
5 vertical hoists.

Workshop and blacksmith's shop

3 lathes (5-ft, 8-ft, and 16-ft).
1 milling machine.
1 radial drill.
5 grinders.
1 trackpin press.
2 fan forges.
1 capstan lathe.
1 planing machine.
1 screwing machine.
1 cylinder-boring machine.
6 electric welding sets.

Woodworking machinery

2	saw benches.	1	bandsaw.
1	planer and thicknesser (24-inch).	1	shaping machine.
1	planer.		

Floating craft, etc.

2 tugs and eight barges for ferrying sand.
5 dinghies and miscellaneous pontoons, etc.
4 sets divers' equipment.

Motor transport

3 40/45 trailers.
4 13-ton rock-bodied tippers,
and a fleet of 5-ton tippers and miscellaneous station wagons, jeeps, land-rovers and staff cars.

A well-equipped garage was established on the site.

Laboratory

A full-time European assistant engineer was in charge of the laboratory, assisted by two Africans, one trained on site as a laboratory assistant, and the other trained by the East African Meteorological Department at Entebbe as an observer.

The laboratory was adequately equipped with testing equipment to cover tests on concrete and cement samples to B.S.12 and B.S.1370. The station was also classified as a second-order meteorological station and was equipped with a standard screen containing max.-and-min. thermometers, wet- and dry-bulb thermometers, and a piche tube for evaporation measurements. In addition two evaporation pans, a standard raingauge, and barometers were provided.

APPENDIX III

STATISTICS RELATING TO CONTROL OF CONCRETE MANUFACTURE

Classes of Concrete Specified

Concrete class	Coarse aggregate : cu. ft	Sand : cu. ft	Cement : cwt	Maximum w/c ratio	Cement : lb. per cu. yd.
6/¾	20	11·85	6	0·50	700
6/1½	20	10·60	6	0·50	696
5/¾	20	13·00	5	0·61	625
5/1½	20	12·65	5	0·58	576
4/1½	20	12·65	4	0·70	475
3/3	20	12·20	3	0·80	369
3/1½	20	13·00	3	0·80	392
6/⅜	20	12·80	6	0·67	624
5/⅜	20	12·70	5	0·66	543

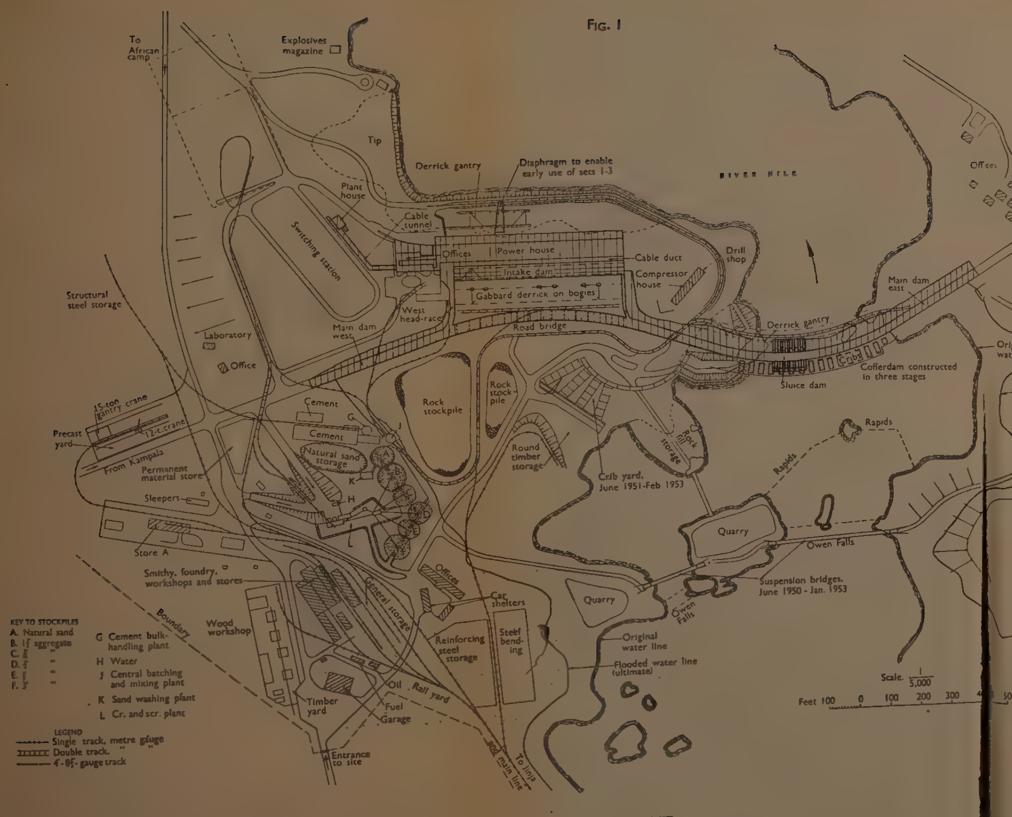
Notes : "Class 5/¾" denotes :—

That ¾-inch aggregate is the top size of stone.

That 5 cwt of cement is to be batched with 20 cubic feet of combined coarse aggregate.

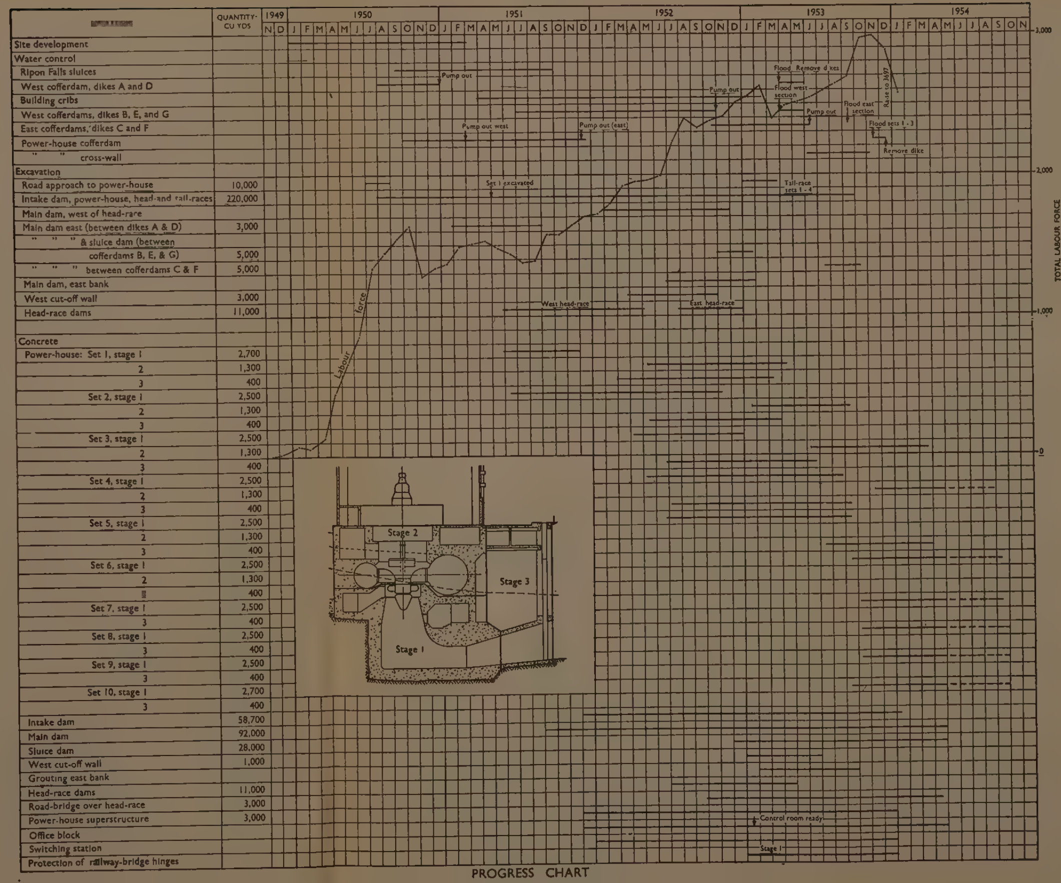
It was found that the maximum water/cement ratios could be reduced for most classes of concrete and that with Tororo (B.S.1370) cement even greater reductions could be made. For instance, if a w/c ratio in excess of 0·54 was used with class 5/1½ the concrete turned out very wet, there was a tendency to segregation, setting time was retarded, and compression strengths at 7 days were often lower than those specified.

Fig. 1



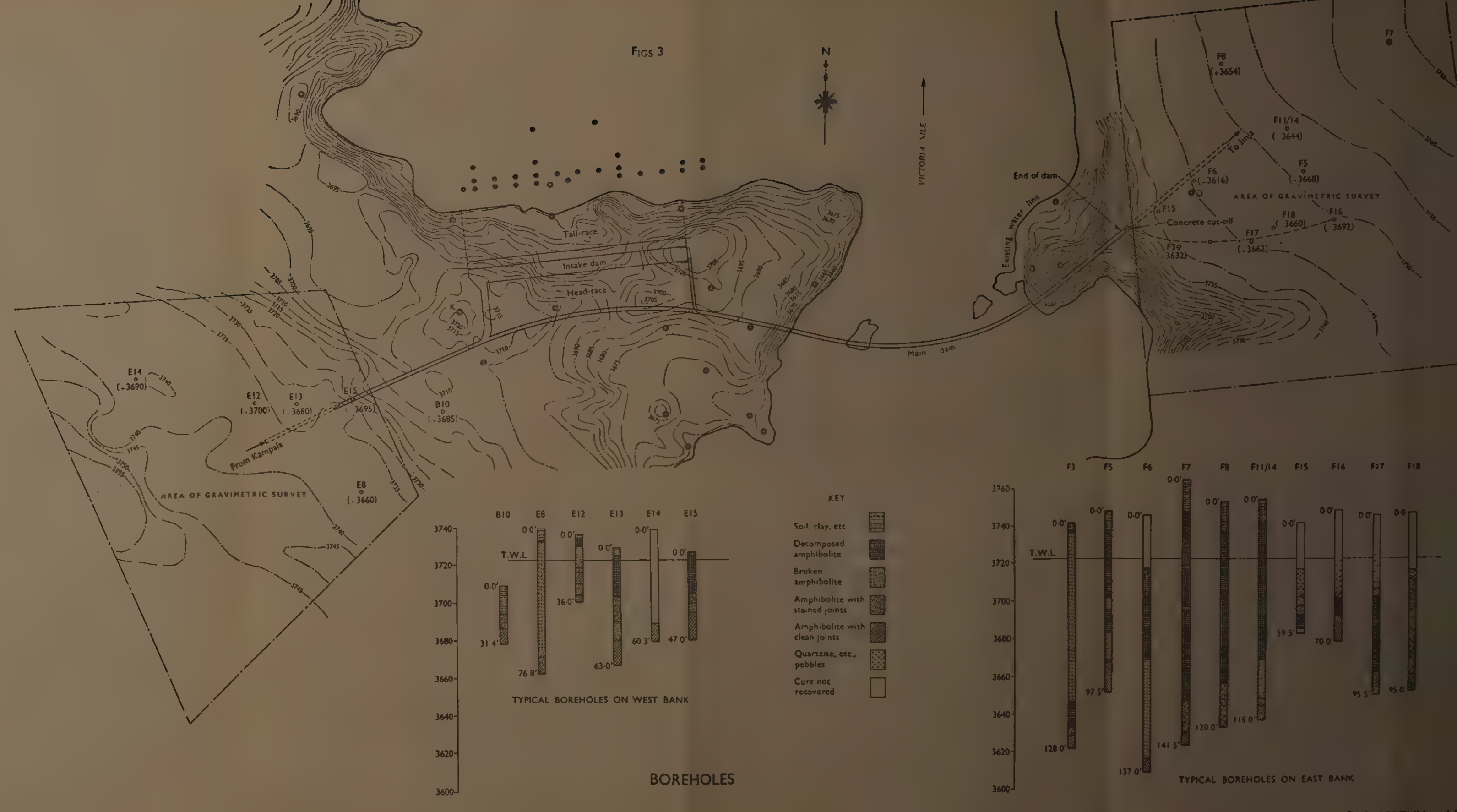
SITE LAY-OUT

Fig. 2



PROGRESS CHART

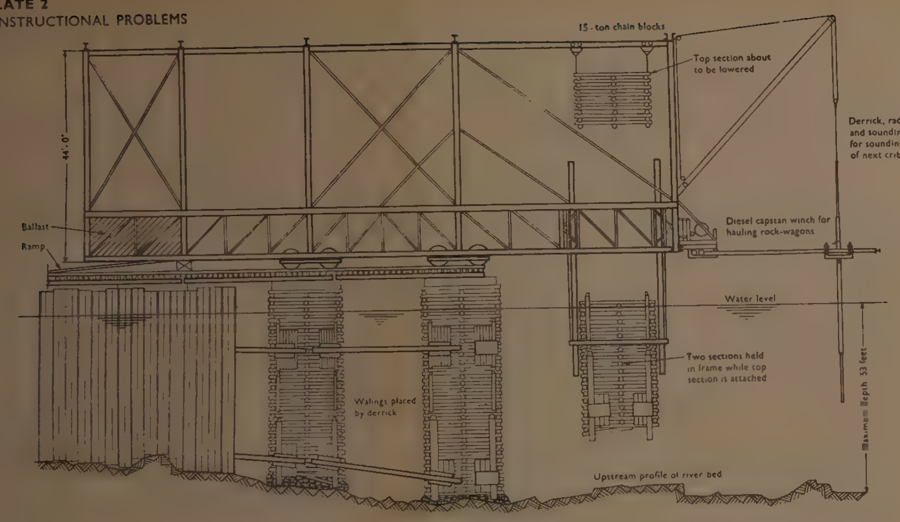
Figs 3



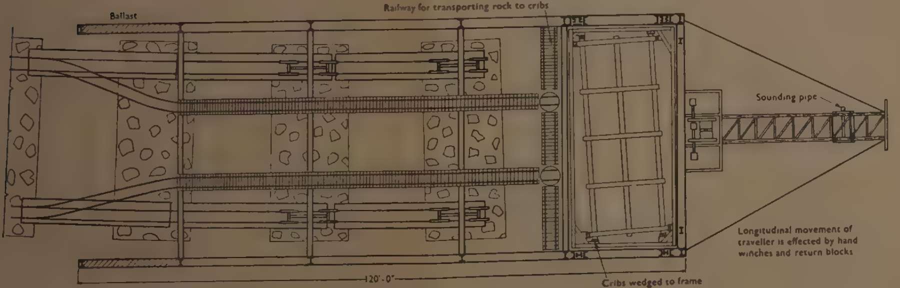
BOREHOLES

PLATE 2
OWEN FALLS: CONSTRUCTIONAL PROBLEMS

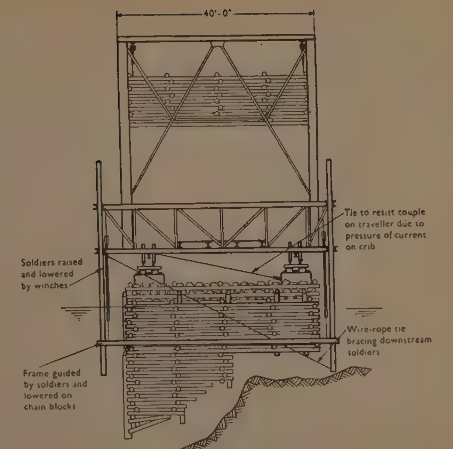
Figs 5


15-ton chain blocks
Top section about to be lowered
44'-0"
Derrick, radial arm, and sounding pipe for sounding area of next crib
Ballast
Ramp
Diesel capstan winch for hauling rock-wagons
Water level
Two sections held in frame while top section is attached
Walings placed by derrick
Maximum depth 53 feet
Upstream profile of river bed

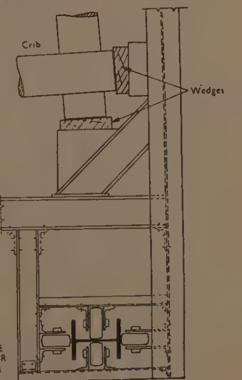
SIDE ELEVATION


Ballast
Railway for transporting rock to cribs
Sounding pipe
Longitudinal movement of traveller is effected by hand winches and return blocks
120'-0"
Cribs wedged to frame

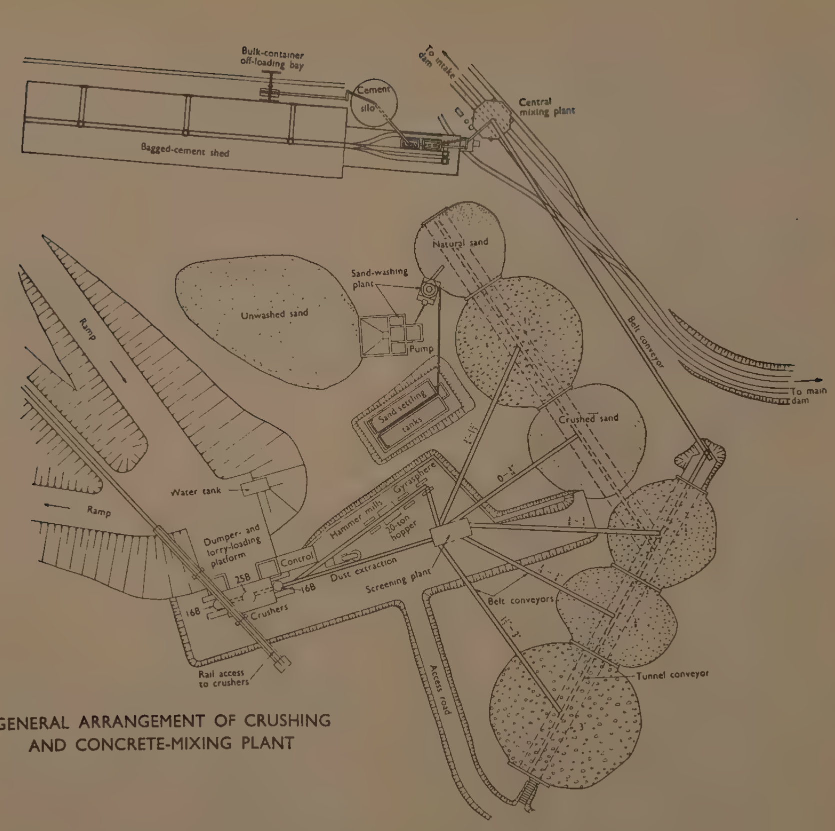
PLAN


40'-0"
Tie to resist couple on traveller due to pressure of current on crib
Soldiers raised and lowered by winches
Wire-rope tie bracing downstream soldiers
Frame guided by soldiers and lowered on chain blocks

FRONT ELEVATION

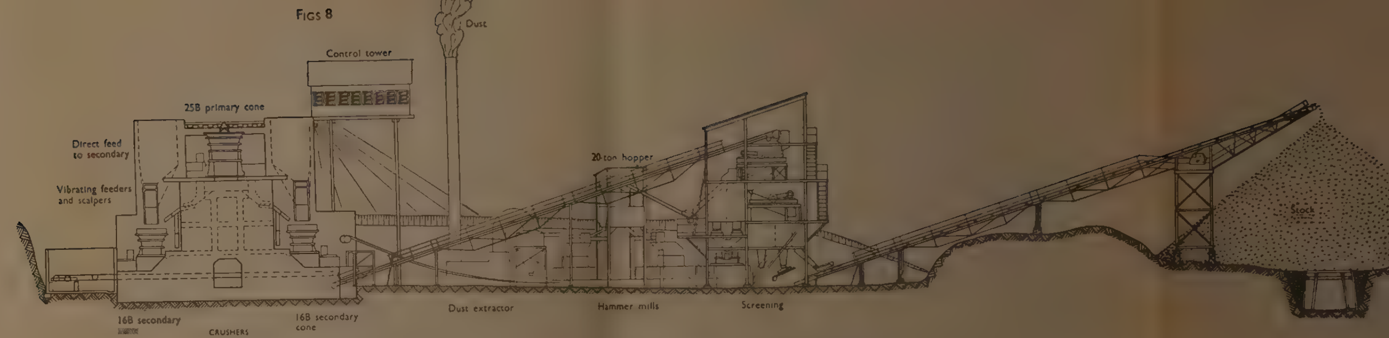


CORNER OF CRIB GUIDE FRAME SHOWING ROLLER MOUNTING TO SOLDIER

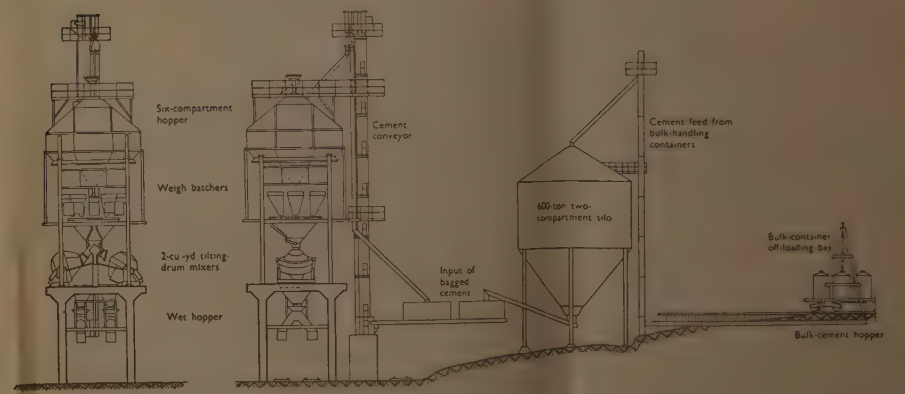


GENERAL ARRANGEMENT OF CRUSHING AND CONCRETE-MIXING PLANT

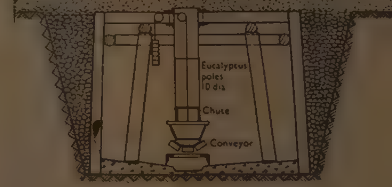
Figs 8



CRUSHING AND SCREENING PLANT


Six-compartment hopper
Weigh batchers
2-cu-yd tilting-drum mixers
Wet hopper
Cement conveyor
Input of bagged cement
600-ton two-compartment silo
Cement feed from bulk-handling containers
Bulk-container off-loading bay
Bulk-cement hopper

MIXING PLANT



RECLAIMING TUNNEL

CONCRETE PRODUCTION

Grading Curves for Coarse Aggregates

The aggregate consisted of amphibolite schist excavated and crushed on site. It tends to flakiness and trouble was experienced with the classes of concrete involving a large proportion of ¾-inch stone. Aggregate crushing tests gave a mean figure of 13·6 per cent.

Comparison of Cube-Test Results obtained using Imported and Local Cements

The results of compression tests on cubes made with imported and local cements are given in Tables 1 and 2 respectively.

Generally it was found that the local low-heat cement over-took imported cement in crushing strength after 6–7 days.

TABLE 1.—TESTS ON CONCRETE MADE WITH IMPORTED CEMENT (ANCHOR, BULLDOG, "PRIMA" and "THIEU" brands) (B.S. 12)

Concrete class	Cube strengths : lb. per sq. in.						
	7 days			28 days			Mean gain on 7 days : per cent
	Mean	Max.	Min.	Mean	Max.	Min.	
6/1½	4,290	5,480	3,020	5,423	G6,220	4,230	26
5/1½	3,243	5,200	1,740	4,260	6,220	2,490	31
5/¾	2,666	4,460	930	4,153	6,190	1,430	56
4/1½	2,646	4,010	1,210	3,651	5,350	1,620	38
3/3	1,946	3,200	1,190	2,660	G3,500	1,780	37

Note :— "G" indicates greater than, that is to say, the cube was not broken at the full capacity of the machine.

TABLE 2.—TESTS ON CONCRETE MADE FROM LOCAL CEMENT (UGANDA CEMENT INDUSTRIES, TORORO)

Concrete class	Cube strengths : lb. per sq. in.						
	7 days			28 days			Mean gain on 7 days : per cent
	Mean	Max.	Min.	Mean	Max.	Min.	
5/1½	3,370	4,090	2,280	5,610	G6,220	3,600	73
5/¾	2,423	3,110	2,020	4,890	5,600	4,320	102
4/1½	3,194	4,260	1,680	5,148	G6,220	3,810	61
3/3	1,792	2,170	980	2,922	G3,500	1,882	63

Discussion

Mr W. J. H. Rennie said it was obvious that the constructional work had been elaborately planned and carefully executed. He was glad to see that the contractors had made use of hydraulic models to assist in the design of the temporary works, since that was a demonstration of the scientific approach to the solution of constructional problems.

The design and construction of the cofferdams and the control and diversion of the river had presented many problems, although the overall difficulties had been simplified by the construction of the temporary barrage and the regulation of the flow at Ripon Falls.

He called attention to the fact that Sir John Aird on the original Aswan Dam, lower down the Nile, had had to contend with more difficult river conditions, both in volume and depth, in the construction of the sudds for the Bab el Kebir at the first cataract.

Mr Rennie wished to compare the Owen Falls cofferdams with those used on the Perak River Hydro-Electric Scheme in Malaya, where he had spent several years and where eight different types had been used. The hydrology of the two rivers was quite different and in contrast to the Owen Falls' regulated flow of 21,000 cusecs, the uncontrolled low-level flow of the Perak River was about 5,000 cusecs; it had been, however, subject to frequent flooding and, whilst the average flood was about 70,000 cusecs, a flood of 300,000 cusecs had actually occurred at the time the contractor had begun operations. The main difference, however, was that all the cofferdams had had to be designed for being overtopped, because to make provision for even the average floods would have been prohibitive both in time and cost.

Figs 21 showed the crib-and-stoplog type of design which had been adopted for the upstream arm of the closing length of the four main cofferdams.

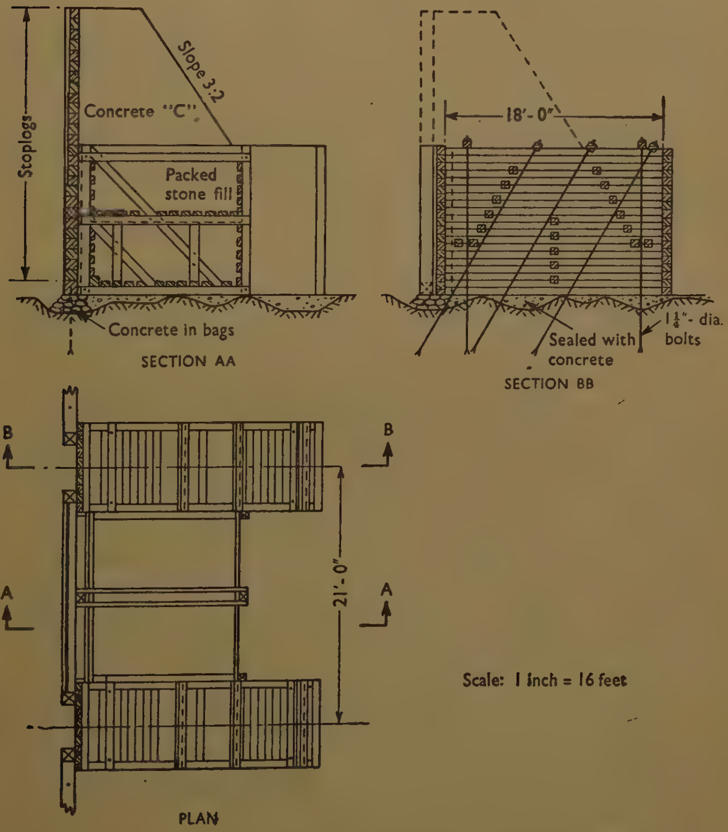
The arm had been about 250 feet long and had consisted of gravity-section concrete abutments and ten cribs, 20 feet long, 8 feet wide, and 25 to 30 feet high, as shown, designed to exclude floods up to 36,000 cusecs. The cribs, which had been made of squared timber, had been constructed on the river bank and launched and floated into position. All the timber used on the cofferdams had been either imported Baltic Pine or Douglas Fir and had been used on other parts of the job at least three times. No provision had been made for tailoring the bottoms to fit the rock surface as had been done with cribs of the previous cofferdams and also at Owen Falls. Immediately the cribs had grounded, they had been fixed and levelled with timber stilts, the ballast pockets filled and the perimeter sealed with concrete in bags placed by divers working inside the cribs; the velocity of the river at that time had been about 12 feet per second.

The cribs had then been anchored to the rock by means of vertical and inclined holding-down bolts, filled with rubble, and grouted through pipes left in for that purpose. A concrete wall had been superimposed on the cribs and that had formed the abutments for the stoplogs which had been placed in position later, the whole of the operation of closing the gaps and diverting the river having been completed in 6 days. One of the advantages of having grouted the cribs solid had been that they only required to be half the size of the loose-rubble type, owing to their increased density.

Would the Authors explain why the cofferdams had not been sited farther away from the dam structure so that they could have had more working room ; the upstream arms especially appeared to be much too near to the permanent work because, for example, any leakage was more readily dealt with inside the cofferdam than by having to go out into the river again for a cut-off, as had been done at Owen Falls.

The cross-sections through the dam and the intake showed continuous cut-off trenches, but the sluice dam (*Figs 6(a)* and *6(c)*) had no cut-off at all, although the rock immediately upstream was about 10 to 15 feet below the foundations of the sluice dam. Could the Authors explain that omission and also why the whole of the sluice dam had not been included in the cofferdam No. 2 ? The sluices could then have been used for the diversion of the river, thus simplifying the construction of cofferdam No. 3, which had given so much trouble and had taken 5 weeks to dewater.

Figs 21



Also the programme had indicated that the sluice dam had taken 18 months to construct and that seemed to Mr Rennie to be a very long time indeed; the construction of the sluice dam in two sections appeared to have been necessary solely because of the proximity of the cribs; and the fact that the upstream section had had to be underpinned would certainly have slowed down the rate of working and added to the cost.

Referring to the consortium, he had difficulty in understanding the advantage of having eight different contractors in contrast to the Perak scheme, where the whole of the civil engineering construction had been carried out by one British contractor.

One of the Authors had recently been connected with the construction of a dam on which cableways had been used and he observed that, in the case of the Owen Falls dam, use had been made of derrick cranes only. Bearing in mind that cableways had been used across the Nile on the barrage at Nag Hamadi and on the dam at Gebel Aulia, would the Authors say why cableways had not been used at Owen Falls and if they had been considered, why they had been rejected?

Professor A. H. Naylor referred to the method adopted for taking soundings in the deep swift-flowing parts of the river.

During the investigation of 1935[1] he had to take some soundings without being able to make much preparation. He had found it possible to row out in a small rowing boat to within about 2 feet of the rapid flow above the Ripon Falls and down near the Jinja bridge, but, obviously, right across the middle much greater preparations were necessary. He had felt that it was necessary to get some information about that part of the river in order to get some idea of the depths to which a dam at the Jinja bridge would have to be taken. A large lifeboat had been sent up from Mombasa. It had been moored to a projecting spur of rock, and about 400 feet of rope had been let out. There had been gadgets for trimming the nose of the boat, and a rudder at the back. An axe had also been provided to drop the anchor at a moment's notice. Most important of all, there had been a long length of warp held by a gang of natives on the bank. In spite of all those precautions, the moment the nose of the boat had entered the main current the boat had begun to swing broadside-on and only the warp had prevented a disaster. After that, he extrapolated the soundings!

While taking the soundings from the boat they had flung the lead upstream, allowing it to swing down and taking the first contact with the rock as the lead passed the vertical. The method of sounding used by the Authors obviously meant that they could not get a vertical sounding, and he presumed that they had had to estimate the slope of the lead.

He was interested in the information about the gravimetric survey.

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[1] Preece, Cardew and Rider, and Coode, Wilson, Mitchell and Vaughan-Lee; "Uganda Protectorate Hydro-Electric Investigation of the Victoria Nile." June 1935.

He gathered that it was very successful. Could some quantitative figures be given to illustrate its accuracy ?

On p. 680 reference was made to the 26-foot gravel pocket and the seams which were in a north–south direction. In the discussion on the preceding evening, Professor Naylor had explained why, in his opinion, the location of the dam should have been at the upper falls, the Owen Falls proper, rather than at the lower site. He had felt that the fact that the river channel was narrower at that point meant that it was naturally deeper. That indicated the probability that the rock was more decomposed and less suitable for a foundation.

He had expected that some seams in the north–south direction would be found when the excavation was uncovered. During the 1935 investigation he had been shown several buildings in the township of Jinja which were always cracking. The buildings would be repaired, and then in the course of a year or so cracks would appear again. The cracked buildings had been plotted on a map, and found to be in a straight line. That indicated an active fault plane. If the line was continued, it met the Nile at the Owen Falls, and thereafter the Nile valley was in the direction of the fault.

Naturally, that had made him rather concerned if a major dam was to be located at the Owen Falls. He had sought the advice of Mr E. J. Weyland, the Director of the Uganda Geological Survey, who had told him that Jinja was not an epicentre and that there had not been any recent earth tremors there, but he would not rule out the possibility of a continuous glide, which was obviously what was taking place. Nevertheless, Mr Weyland had said that a fault had been traced and that the agreement between the location of the fault and the observed cracks was too close to be a mere coincidence. He had gone on to say that there was no indication of the existence of a fault in the bed of the Victoria Nile near Jinja. Professor Naylor had taken Mr Weyland down to the Owen Falls site, and they had discussed the matter ; Mr Weyland had been quite confident about the project. Conclusions to that effect would be found in the 1935 report.[2] Mr Weyland being quite confident, Professor Naylor had felt that it was not for him, a layman, to question the correctness of Mr Weyland's attitude, and, therefore, he thought that it was safe to recommend the damming of that part of the river. Nevertheless, he had felt that it was strange that an active continuous glide should be taking place in Jinja and that only 2 miles farther down it should have ceased altogether.

He had expected therefore that one would find seams running in a north–south direction in the bed, and not only that, but that careful observations might show a very slight creep. He could not think on what grounds Mr Weyland had derived his confidence since he could have had no opportunity of inspecting the bed of the river. Professor Naylor was very interested to know all about the north–south seams in the river now that the contractors had had the opportunity of uncovering them and of observing exactly what they were like. He wished to know whether they had taken very careful measurements to find out whether there was even a microscopic movement there. He would have regarded them with great suspicion. That was one of the reasons why he had favoured the upper location for the dam. He felt that with the much lower dam which would have been necessary at the Owen Falls proper, it would have been much easier to design a special slip joint—it would be difficult in any case—which would take charge of any differential movement of the two sides.

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[2] *Ibid.*

With regard to the Tororo cement, he was interested to note that the contractors were so successful with it that not only could they obtain the same strength in a month as with normal Portland cement but it was also a low-heat cement. He would have expected it to be less strong as a result of the effort to obtain the low-heat quality; would the Authors say whether it was equally finely ground and to what extent it was a low-heat cement?

Mr H. J. B. Harding said that in January 1954 he had paid a flying visit to Uganda and Kenya to see works which his firm were doing there, and had been able to visit the works at Owen Falls.

They gave at first a sense of incongruity. That pioneering work in the heart of Africa was in the suburbs of a vigorous but rather unattractive town, with modern "gentlemen's residences" on the road to the dam, which was fed by a main-line railway and on a main motor road. The first impression was that the work was much closer to civilization than any valley in the Scottish Hydro-Electric Schemes.

However, on second thoughts one remembered the diseases, the flies, and the crocodile infested water; and whilst it was nice to be on a main-line railway, much depended on where the other end of the line was situated.

In giving credit to the consortium of companies, the effect of seeing only three British names should be countered by remembering that the Owen Falls Dam, to some extent, had been built by the British Engineers who also built Mombasa Harbour and the Uganda Railway from 1900 onwards. If it had not been for their work the Owen Falls Dam could not have been built in an already somewhat civilized area.

To counter those advantages it should be remembered by future readers [of these Proceedings] that the builders of the Uganda Railway in the early 1900's had strung out a telegraph line as they went, so that, when 750 miles inland they required additional plant, all they had to do was to send a telegram to Great Britain and it would arrive at the rail-head, possibly 6 months quicker than it would in the conditions of 1950 and after.

The fact that English newspapers, and even directors, could be flown to the site in 2 days from England was a poor consolation for the long periods of waiting for materials to accumulate.

The Owen Falls Company had very wisely made good labour-camp provision and had looked after their workers properly. Mr Harding had afterwards seen the new native hostels at Welkom, in the Free State goldfield, which were a great improvement on the old-fashioned Johannesburg compound, with a high standard of accommodation and welfare, and with kitchens and hospitals which were envied by the Europeans; they were an answer to those who talked of native exploitation without seeing conditions for themselves.

It saved much time and trouble if the promoters of large schemes built the roads and camps before contractors had to start work, since they would often be built by local talent, with entirely different equipment from that needed for the heavy construction work. That was said to be the intention of the promoters of the Kafue Scheme in Northern Rhodesia.

The lay-out of the working site was of great importance in any large construction scheme, and was well shown in the Paper. So little information had been given in the past that it would be worth while filing all the different site lay-out plans which had appeared in Papers presented to the Institution.

On the Owen Falls site two 2-cubic-yard mixers had been used. On several recent dams, such as the Oberaure Dam and some Scottish dams, plants containing as many as four 4-cubic-yard automatic mixers had been used. Had the Authors found that two mixers were adequate?

The list of plant given in the Paper was very helpful in showing how much was required for such a work.

What was the Authors' true opinion on the use of cyclopean masonry, or "plums?" There were great arguments as to whether they were worth the trouble.

He complimented the Authors upon using so many derricks. Subject to economics one could not have too many on such a site.

Cableways had their uses in certain cases, especially for high dams, and were used with success on jobs such as Loch Sloy, but ever since on each tender invited by the Scottish Hydro-Electric Board for another dam, they were still peddling the cableway which they had provided. Mr Harding's firm had used a cableway of 900-foot span in 1935 and 1936 for excavating the clay pit in Chingford Reservoir, and it had then lain in the long grass for years with no further suitable use occurring. Derricks were of universal use, on the other hand, and those who had ever used really powerful derricks would always tend to return to them for most problems.

Mr C. G. Chatham said that shuttering was a very thorny subject. A cry had gone up for carpenters and yet more carpenters; but the steel shuttering was quite satisfactory, and so was the resin-bonded plywood.

The local timbers, however, had given a lot of trouble. What would the Authors do in that respect if they were faced with the task of constructing the next dam down the river?

Dr Olivier, in his Introduction, had said that there was difficulty in getting the concrete underneath the curves of the turbine scroll casings. That had been true at Owen Falls. Mr Chatham believed, however, that a grouted concrete could be used. It had been done very successfully in the United States. Consideration had been given to its use at Owen Falls, but they had decided that they were not ready for it, and rather than chance not making it a complete success, they had not used it. However, it was now known that it could be done with complete success.

Turning to the crushing plant, Mr Chatham referred to the section on "Concrete Production" on p. 693, and to Figs 8, Plate 2. One very small detail in relation to the plan in Figs 8 was that there ought to be a small conveyor leading from the screening plant to the 20-ton hopper. The plant could be criticized in that it was a very expensive one. The f.o.b. value was £56,000 or thereabouts, but, of course, one had to bear in mind that it had to operate a long way from home. The rated output was 200 tons per hour, and it was producing 3-inch size down to dust. Ten per cent of that was sand, with the result that the output had been 180 tons per hour of coarse and 20 tons per hour of fine. The estimated requirement was 100 tons per hour, and it had been reckoned that that would be doubled so as to allow for double shifting and any possible periods of breakdown.

The central mixing plant had been entirely automatic, although previous experience had been with non-automatic plant. The absolute maximum output of the mixers was 120 cubic yards per hour, but of course, it had been impossible to achieve that, because the concrete could not be distributed and placed at that rate. The actual output had been about 80 cubic yards per hour.

With men working 24 hours round the clock (and that had been envisaged) it had been decided to make the crushing plant very large so that concrete could be mixed throughout the 24 hours if it was required to do so without having to crush stone throughout the same period. The 25B dealt with 350 tons per hour with a 4-inch opening and could deal with almost any stone that the excavator could pick up, so that there had been no need to select the size of stone with which to feed it.

A small stock had been provided for underneath the 25B, and that had been fed to the two 16B's over two electric vibrating grizzlies in which ⅞-inch or ⅝-inch screens could be fitted to take off any muck present. The 16B's had been installed because if there had been a bad breakdown with the 25B the contractors would have continued by using the 16B's as primary crushers. The screens had had five decks for separating five sizes, and the sized stone had fallen into bins below. Five sizes had been chosen in order to facilitate strict control over the batching of the concrete.

The problem of a shortage of $\frac{3}{4}$-inch material arising had been foreseen, and in an effort to overcome that a 48-inch " Gyrasphere " crusher, fitted with a fines bowl instead of a standard one, had been incorporated in the lay-out. Its largest opening had been $3\frac{3}{4}$ inches.

One noteworthy feature of the lay-out had been the arrangement to bleed off the 3-inch and the $1\frac{1}{2}$-inch material by means of electro-magnetic vibrating feeders, which were adjustable from zero up to maximum output. One could divert either the whole supply from the bins or only a trickle, according to the requirements of the stockpiles.

The natural, rather finely graded, washed lake sand delivered from the cone to the stockpile had emerged as a slurry, and, as such, it had been very wet for the tunnel conveyor to pick up. A twin chute had been installed, and sand had been drawn from the heap which had been formed longer, and which was therefore drier than the other.

The whole plant had proved to be very reliable and had produced what was required. Flakiness had been one of the great troubles anticipated and had, to some extent, occurred ; hammer mills had been installed to rectify it, otherwise they would have been omitted for they were costly items of plant to operate. Their use was kept to the minimum.

Mr Savile Packshaw said that many difficulties had been overcome in the construction of the Owen Falls dam, among them the construction of the cofferdams which were supported by rock-filled timber cribs carefully tailored to the profile of the rock bed.

The use of timber cribs for supporting the sheet-piling had been made possible, or at any rate facilitated, by the fact that the dam was perched along the top of a ridge of rock. Therefore, the rock under the crib sloped upwards and away from the sheet-piling and there was no danger that the cofferdam might be forced down into the excavation for the dam proper.

The presence of fairly hard rock at the surface of the river-bed, or at any rate very close to it, had ruled out, he presumed, the alternative of the self-supporting cofferdam consisting of two parallel lines of steel piling connected by tie-rods and containing filling of rock or other suitable hard material. A cofferdam of that nature depended upon its mass to act essentially as a gravity structure and it usually had a width of about two-thirds of its height. However, in that type of dam the foot of the piling must penetrate into the ground in order to ensure that it was securely held. If that type of dam had been considered by the Authors before the work was commenced, he assumed that they decided against it because the necessary penetration of the piling would be impossible to achieve, or at any rate could not be relied upon.

The simplest form of cofferdam for the conditions at Owen Falls was probably that which was generally known as a cellular cofferdam. That consisted of a series of adjacent curved or circular cells made of sheet-piling and filled with rock or other material. The cofferdam resisted the water pressure by acting as a gravity structure, rather like the double-wall cofferdam which he had mentioned earlier, but it was not necessary for the steel piling to penetrate the rock to ensure stability. Unlike the piling for a double-wall cofferdam, the piling for a cellular dam was not subject to bending stresses. Instead of that, it had to stand circumferential tension due to the internal pressure of the filling. The type of piling required for that form of construction was generally known as straight-web piling and at present the material was only obtainable from the United States and certain Continental manufacturers. That might have been one of the reasons why the cellular cofferdam had not been adopted.

The removal of the filling from either of the types of cofferdam which he had mentioned would not, he thought, have presented much more difficulty than the disposal of the rock from the timber cribs which were actually used.

The Authors had made it clear that there had been no attempt to drive the steel piling into the river-bed in order to obtain a watertight seal and along most of the cofferdam a second line of piles had been driven which apparently served the purpose only of providing an area of calm water within which the blanket of murram had been deposited. From the borehole tests, it seemed that the top of the amphibolite rock was decomposed and if that also applied to the bed of the river one would assume that the rock was penetrable, so that if a determined attempt had been made to drive the pile into it with a hammer of sufficient size, the attempt might have been successful. Would the Authors comment upon that ?

Mr R. G. T. Lane commented upon the east bank cut-off. Following the results of the boring and grouting test and of the gravimetric survey, a 70-foot-deep trial pit was excavated at the location of the end of the dam. Not only had that allowed visual inspection of the rock but it had also been used for a " pump-in " test which provided some of the evidence on which the decision to consider the east abutment as an earth and rock-filled dam had been based. The river bank was steep both upstream and downstream of the dam. Rock excavated from the third cofferdam had been tipped against the bank upstream of the dam to provide protection against erosion. On the downstream side a drainage cut had been made on the line of the shortest seepage path. That had been back-filled with a graded filter of sand, gravel, and broken stones, and terminated in a manhole by means of which it would be possible to gauge the amount of seepage. The principal function of the grouting had been to ensure that the seal between the concrete and the earth was efficient.

It was interesting to note that the cost of the exploratory work in connexion with the east bank cut-off had been about £5,000, whereas the estimated saving in cost compared with the conventional concrete cut-off taken down to sound rock was about £150,000.

Mr Bertlin, in reply, thanked Mr Rennie for his interesting contribution and particularly for the description of the scheme for the Perak River dam. At Owen Falls, most of the cribs had been left in position, and so there the problem of removing the rock from the cribs had not arisen.

It would have been difficult to adopt Mr Rennie's method at Owen Falls because of the very uneven bottom, which shelved steeply, being much deeper on the upstream than on the downstream face. The idea of holding-down bolts was interesting, but at Owen Falls they were not sufficiently certain of the foundations to have depended on such bolts. One pocket, about 25 feet deep had posed a very difficult problem. The fact that it had been possible to place the cribs, which were considerably larger than those used in Mr Rennie's scheme, at the rate of one crib in 8 days, showed that it would have been difficult to improve on the method adopted.

The siting of the sheet-piles and the cribs so near to the dam had been governed by the rapid shelving of the bed upstream of the dam. The distance from the dam to the cribs had been kept to the minimum in order to keep the length of piles and height of cribs also to the minimum. In the original scheme it had been the intention to place the piles and the cribs considerably farther upstream, in order to include the sluice dam without any underpinning, but after a more detailed investigation, it had been found that the depth of water farther away from the dam was such that it would be better to face the difficulty and tortuousness of the underpinning process, and he believed that they had been correct in doing so.

The choice of position of the cross-wall had been a question of balancing the different sections of the work. Four sluices had been sufficient for control in the early stages, whilst the Ripon Falls sluices were in operation. In order to commission the first turbine as soon as possible, all the construction had been subordinated to carrying out the work necessary for raising the upstream water level. An early start on the construction of the first four sluices had therefore been of great importance, and since sluices Nos 5 and 6 had not been necessary at that stage, the cross-wall had been placed in a position to exclude them.

With regard to the time taken to build the sluice dam, the progress chart given in the Paper might be somewhat misleading, because the line showing the progress in constructing that dam included the work in different cofferdams and it did not bring out the fact that work had been started on the foundations of sluices Nos 1 to 4 long before it could be started on sluices Nos 5 and 6. He did not think any time had been lost in that work. The speed of construction of the sluice dam had been dependent on the completion of the cofferdams, and also the gate guides, rather than on the construction of the sluices and the dam itself. Gate guides had to be placed very accurately, and that type of construction had to be done very carefully and deliberately and so necessarily took considerable time.

Mr Rennie's remarks regarding cableways *versus* derricks interested him not a little because, after cableways had been chosen for another construction scheme, Mr Rennie had asked him why derricks had not been chosen. Now he was asking why they had not used cableways. The two systems were different in many respects. The chief reason for the choice of cableways in the case of the previous construction was that there were steep banks and the dam was very high ; those conditions lent themselves to cableway construction. The width of the dam also made the use of derricks difficult. He knew that Mr Rennie had a very ingenious solution for that problem, but in Mr Bertlin's opinion the cableway construction was the better choice and he would adopt it if he had to do that job again. In the case of the Owen Falls dam, cableway construction was really out of the question because the towers would have been quite monstrous, and the various operations did not in the least lend themselves to the use of cableways.

Professor Naylor had raised the question of a fault. When the cofferdams had been completed and the foundations excavated, it had been found that the latter were quite sound and showed no sign of a fault. The pockets of loose material with which there had been so much trouble were not entirely unexpected. They were a common geological feature. Mr Bertlin had discussed the question with Dr Davies, formerly the Director of Geological Survey of Uganda, who had said that it would have been surprising if they had not been found. In a river such as the Nile, with a very uneven bottom composed of various types and hardnesses of rock, the areas of weak rock were scoured out and so pockets were to be expected.

Mr Harding had asked for an opinion on the use of indigenous labour. The output of the African was disappointing and it had been hoped to find more leadership among the men employed. On the other hand, quite a number took to various skills, particularly in the blacksmith's shop and in elementary fitting work, and some showed great aptitude for a number of semi-skilled jobs such as loco- and dumper-driving. The African seemed better adapted to that sort of work than to craftsmanship such as carpentry, where more individual initiative was required.

The discussion regarding the size and number of mixers, a point also raised by Mr Chatham, depended not only on their output, but also on the fact that the job was in the middle of Africa and there had to be some assurance against delays by breakdowns. Another point was that, from time to time, the mixers had had to be relined. Having two mixers gave reasonable assurance of continuous operation and, in fact, they were never out of commission together. The mixer-capacity of 2 cubic yards selected itself, because 2-cubic-yard skips were used—a suitable size for use with 10-ton derricks. Had a smaller size been chosen it would not have been possible to make full use of the derricks. Provision had also had to be made for a peak period, which had lasted 3 or 4 months, when the maximum output of the mixing plant had been used.

Mr Bertlin did not favour the use of cyclopean concrete if speed were of importance, for the placing of the cyclopean blocks upset the even rhythm of concrete placing and that should be fully taken into account in any calculation of the possible theoretical saving in cement.

The number of derricks used was a question of progress, and he considered that the correct number had been chosen to enable rapid progress to be made. They were very useful tools, as Mr Harding had said, and were far more adaptable than cableways ; for example, they could be used much more easily for lifting shutters and for many other operations.

Mr Chatham had referred to the difficulty about using local timber for shutters. Mr Bertlin agreed that it had given a great deal of trouble. The local timber was not really suitable for construction purposes, but it was very much cheaper than the imported variety. In view of the experience gained at Owen Falls, for any future work in East Africa, he would seriously consider importing timber from Europe, Canada, or elsewhere.

The possibility of using pre-packed concrete around the spiral casings had been mentioned by Mr Chatham. The method which had been used with ordinary concrete and grout was quite satisfactory. Mr Bertlin had considered the possibility of using either pre-packed concrete or Colcrete but, in his opinion, either would have worked out more expensive.

Mr Chatham had also mentioned the question of the flakiness of the stone. When the choice of aggregate had been under discussion that aspect had been investigated, and, as mentioned by Mr Chatham, it had been decided to install hammer-mills to obtain a more cubical product. In practice, it had been found that the Gyrasphere as a tertiary crusher was adequate and it would have been better not to have had the hammer-mills, which did not give such a good output and were very expensive to maintain, particularly for the reduction of such a hard rock as amphibolite. Although the concrete had not been easy to work, vibration had been used throughout and, in places, a wetting agent also (Lissapol-N) which had enabled a dense and strong concrete to be obtained.

Mr Packshaw had referred to the question of using two lines of piling to form the cofferdams. The obvious difficulty of placing tie-rods as well as the difficulty of driving the piling had militated against their use. With regard to the use of cellular dams, there would have been great difficulty in driving or placing such dams in such a fast-flowing river. There would also have been the difficulty of building guides and working platforms. The method which had been adopted at Owen Falls was simple and effective. Mr Bertlin did not think that the speed of forming the cofferdams could have been improved. The delay had occurred when dealing with the rock below the dams.

Driving piles into the river-bed was not possible. They would penetrate an inch or two, but it was not a question of better and heavier hammers because they would only have resulted in damage to the toes of the piles. In fact, in certain cases they had done so. Although it was called decomposed amphibolite, the rock in the bed of the river was still very hard and to drive piles into it had not been practicable.

Dr Olivier, in reply, said that Mr Rennie had raised the question of the cut-off under the sluice dam. In the drawings to which he had referred the sections showed projections in front of the main dam, and therefore in front of the main cut-off line. He would like to explain, however, that the exposed rock in that area had proved to be generally clean and sound, and in some instances it had actually been possible to reduce the cut-off depth considerably, provided there was the certainty of being able to grout the foundations.

With reference to soundings, two methods had been adopted. At the railway bridge, where it had been essential to get accurate readings in fast-flowing water, the lead wire had been kept vertical by means of a straining wire attached to the weight, which passed over pulleys carried on a wire tensioned across the river just above the surface upstream of the bridge and actuated from the railway bridge where the soundings were actually taken. At the dam site, however, special precautions had not been taken and it had been necessary to make allowances for the slope of the lead line caused by the current.

Dr Olivier then amplified the remarks made by Mr Bertlin in reply to the question of a fault at the dam site. No definite evidence had been found of the presence of a fault in the foundations but one fissure running obliquely across the foundations contained a soft material which had at first appeared to be soluble in water. After excavation of the soft material the fissure had been found to be no deeper than 9–10 feet and, after clearing out the cavity, the foundations had been successively grouted.

The following figures were given to illustrate briefly the comparison between imported and the low-heat Tororo cements :—

Imported cement . . .	6 per cent retained on 170 sieve
Tororo cement . . .	2 per cent retained on 170 sieve

Class 3/3 concrete poured in 40-foot-long blocks :—

Imported cement :

Temperatures rose to 117° F. in 50 hours and returned to 100° F. in 10 days.

Tororo cement :

Temperatures rose to 110° F. in 70 hours and returned to 100° F in 7 days.

Dr Olivier was grateful to Mr Harding for the tribute which he had paid to East African Railways and Harbours ; both Authors wished to be associated with those remarks.

With regard to the finish of the dams, from the purely aesthetic point of view there was no doubt that a slight roughness in surface improved the appearance, particularly when viewed from afar. Provided the concrete quality was good there was certainly no need for unnecessary buffing. A good finish had been obtained from the start on the upstream faces using vibrators, but it had taken a little time to train African vibrator operators to obtain a good surface on the downstream faces, possibly because the vibrated material had tended to fall away from the sloping faces.

Mr Chatham had referred to grouting under the spiral casings. It should be explained that one factor which had influenced construction considerably in the early stages was the necessity to complete set No. 1 as soon as possible, owing to the urgent necessity to reduce thermal power generation on account of the high fuel-costs in Uganda. By the time work underneath the spiral casing had come to be planned in detail the small space for the customary loading bay had been fully occupied by the plant contractors, who had been hard pressed to install their plant. In the case of set No. 2 it had again been necessary to expedite work by normal means on account of the imminence of the visit of Her Majesty the Queen. It had therefore not been pôssible to experiment with pre-packed concrete but it was hoped to try those methods when future sets were being installed.

Mr Packshaw, in his closing remarks, referred to river-bed conditions. When the cofferdams had been dewatered and the rock surface examined a condition not unlike cavitation had been found, the rock being pitted by numerous holes. The conclusion had been drawn that the whole river-bed had been subjected to intense aeration at those falls and that that was responsible for the condition. Except for the top inch or so, however, the bed had not been "soft" and it had been impossible to drive a pile into that material for more than a few inches.

Professor Naylor had asked for quantitative details of the gravimetric surveys. The objects of the survey could be summarized as followed :

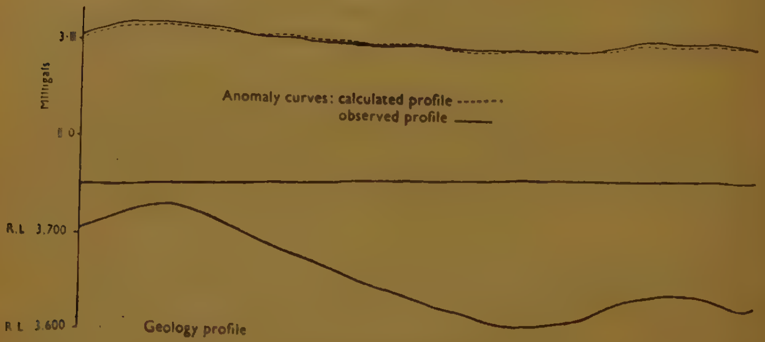
Information obtained from early borehole investigations carried out in the areas of the cut-off walls had indicated that levels of the surface of the solid rock varied considerably. The need for further investigations had been emphasized by the great depth at which solid rock had been found in some places ; but it had been evident that the cost of exploratory work to such depths would be considerable and it had been thought desirable to employ some independent method of locating, or at least of suggesting, a more favourable line for the cut-offs.

Although gravimetric surveys had been used for many years to obtain information on the variations in geological formation at great depths, it had not been possible to benefit from previous experience in the surveys carried out at Owen Falls, since the depths involved were insignificant from the geological standpoint.

Gravity readings taken at different stations would record horizontal variations in the overall density of the material underlying the stations (extending deep into the earth). Any geological condition which resulted in a horizontal variation in density would cause a horizontal gravity variation or gravity anomaly, and the magnitude of the anomaly depended on the density contrasts occurring over the area. At Owen Falls, sound amphibolite (assumed to be of constant density) had been known to exist at a geologically shallow depth, and it had been inferred that variation in gravity readings would therefore give an indication of the variation in depths of the sound amphibolite. By reference to existing boreholes in the area, it had been noted that the maximum known variation of those depths was about 50 feet, yet the variation in gravity anomaly was only about 0·15 milligals (see *Fig. 22*). It would be dangerous to attempt to correlate the gravity anomaly too closely with the depth of sound rock, but that suggested that the anomaly obtainable with the comparatively shallow depth of rock was very sensitive to changes in that depth.

That emphasized the importance of site conditions being favourable for computing the corrections applied to the observed gravity readings. All the corrections also involved the assumption of a value for the density of the overlying material included in the correction and the effect of any error in that assumption would be proportional to the amount of surface slope and " topography " which had to be corrected for.

Fig. 22



TYPICAL PROFILES

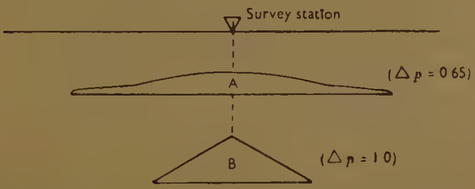
Gravity measurements were sensitive to horizontal variations in the overall density of the underlying material (including rock) but they could not distinguish between the different densities of successive strata : a given gravity distribution could be produced by a variety of mass distributions. It was evident, therefore, that the gravity anomaly map could not of itself provide a geological profile ; other information, such as that given by trial bores, was necessary.

Fig. 23 illustrated how the same comparatively wide gravity anomaly could be accounted for by either a wide shallow density anomaly (A) or a much narrower and deeper one (B) ; or by a variety of density anomalies intermediate between (A) and (B). Nevertheless it could be inferred from the gravity anomaly map, even if unsupported by existing borehole information, that areas shown as gravity " highs " were likely to be more favourable for siting additional trial bores than " low " areas.

When an attempt was made to interpret the gravity anomaly map gravitatively, allowance had to be made for "end corrections" to take into account the fact that any geological profile adopted did not in fact extend indefinitely in the plane normal to the profile. That necessarily involved approximations and assumptions, and the further the surveyed area extended in a direction perpendicular to the section considered, the more reliable those assumptions would be.

Dr Olivier quoted a typical conclusion relating to the investigations carried out on the east bank: "From the results obtained there appear to be gravity 'highs' in the N. and S. portions of the area surveyed, i.e. along lines 4/7, 5/3, and 5/4. These may be quantitatively interpreted as areas where the decomposed rock thins out. The map illustrates these 'highs' (it may be read as if it were an ordinary Ordnance Survey Map). It is thus seen that the boreholes F6, F11, F7 and F8 lie in a small depression in the non-decomposed rock. F5 lies on the side of this depression, this may account for the broken amphibolite at 45 feet. A quantitative interpretation has been attempted but the order of anomaly and the order of error due to terrain approximations, etc., are about the same. However, the accompanying profiles give an indication of the approximate thickness of decomposed amphibolite."

Fig. 23



In general, the results of such gravimetric surveys should not be acted on normally unless confirmed by independent means. In the absence of that, the results of gravimetric surveys could be "calibrated" by utilizing information obtained from existing boreholes.

Correspondence

Mr A. W. Shilston said that he was interested in what might be called the strategic approach to the undertaking. He noted that the contractors consisted of eight firms, British, Danish, and Dutch and, although a coalition of contractors was a practice adopted elsewhere, nevertheless, such an arrangement was, he believed, a comparatively rare practice amongst British contractors—especially with such a large number of participants.

Without encroaching on the commercial relationships between the members of the consortium, he thought it would be valuable if the co-author who represented the Owen Falls Construction Company could give some indication of the advantages of such a coalition and whether the arrangement proved entirely satisfactory from the technical and site administrative viewpoints.

The planning and direction of an engineering project of the scale of the one in the Paper was rather comparable with that required for a military operation. History had proved that with coalitions successful operations in the latter category could not be carried out under committee control. That lesson had been observed during the 1939–45 war when the necessity of establishing Supreme Commanders in the various spheres of operation with wide terms of reference had been realized. In the aggregate, that system proved successful.

Developing the analogy, Mr Shilston wondered whether any difficulties had arisen in the construction of the Owen Falls Dam as the result of possible conflicting ideas and temperaments on the part of the various nationals representing the firms in the consortium.

In planning the operation, had, for example, definite responsibilities been allocated to particular firms or were resources and staff completely pooled ? Although great tact would obviously be called for, had the agent been in effect a supreme commander over all the participant bodies with authority to take major decisions without involving consultations between the headquarters of the various firms ?

Mr Bertlin, in reply, stated that the partners in the firms comprising the Owen Falls Construction Company had originally worked together immediately after the war on the reconstruction of Rotterdam harbour, when the consortium had been known as " Havenherstel Maatschappij." Because that group had operated successfully in Holland, it had been decided to continue to collaborate, and to tender for the Owen Falls contract.

There were several advantages in a consortium undertaking the construction of a large scheme. It enabled the financial risk to be spread, so that no one partner had too many eggs in one basket. It was also of advantage to the client, for he had the guarantees of several firms, both individually and severally. An interesting point in the case of Owen Falls was that, since one of the firms was a nationalized company (Dorman, Long & Co. Ltd.), the client had had, in effect, the backing of the British Government. In the case of a contract being let on a cost or target basis, that advantage did not apply.

Another advantage was the pooling of the resources of a number of organizations. It might be that technically one company had special experience of a particular aspect of the work. Co-operation in the provision of staff might also be an advantage.

Mr Shilston's analogy of a military operation and the necessity for a

Supreme Commander, was apt. A consortium should appoint a policy-making committee which would normally consist of one Director from each firm. That would be the channel through which the Agent could obtain decisions on broad matters of policy and enlist the help of the individual firms.

In the case of the Owen Falls Construction Company, partnership meetings had been held frequently in the early stages, but later about once every 3 months. At those meetings had been decided such questions as whether any particular firm should be responsible for a particular aspect of the work. For example, one firm had advised on the design of the coffer-dams, another on that of the crushing plant, whilst a third had undertaken to handle all the shipping arrangements for plant and materials in Europe. But the responsibility for direction and progress had been placed fairly and squarely on the Agent and he had been given a free hand to plan the work, appoint his own staff, and order the necessary plant and materials.

Human nature being as it was, and bearing in mind that contractors on the whole were strong individualists, it would be idle to expect that there would not be occasional stresses and strains in the organization, and Mr Shilston was correct in his reference to the necessity for tact. However, on the whole the arrangement had worked adequately.

For the construction of major schemes it would seem to be a good arrangement for a consortium of three or four firms to undertake the work, provided their experience was sufficiently diverse to cover all the technical aspects involved.

Correspondence on this Paper is now closed.—Sec. I.C.E.

Paper No. 5986

"Strain Measurements on the Temporary Road Deck for the Toronto Subway"

by

William Robert Schriever

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

Strain measurements were made on the deck beams and some other parts of the temporary steel structures of the street deck used during the construction, by the cut-and-cover method, of the Yonge Street Subway in Toronto, Canada. The results of measurements on the various deck beams are presented.

The investigation included both normal traffic loads and heavy load combinations. The maximum values of stresses and the frequency of occurrence of loads are discussed, with a view to improvement in similar designs which may be used in the future.

INTRODUCTION

In using a certain allowable stress in the design of an engineering structure, an engineer usually visualizes the factor of safety as the ratio of the stress at failure to the allowable stress with which he is designing. He may also think of another factor of safety with regard to yield, namely, the ratio of the stress at the beginning of inelastic yield to the maximum stress occurring in the critical points in the design of the structure. He knows that values of about three and two respectively, are used at the present time for these two factors of safety for structural steel. The engineer should consider whether the above values actually represent the factors of safety of the full-scale structure, and further, what this factor should be.

Professor Pugsley[1] has reviewed some of the many considerations entering into the question of safety in structural engineering, and has pointed out that the " stress factor of safety " was introduced and connected directly with early material testing which, in a new and unprecedented way, provided numerical values for the strength of various materials. Previously, only " load factor of safety " had been used, that is, the ratio of the load required to break a girder to the greatest load the girder would actually have to carry.

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† Correspondence on this Paper should be received at the Institution by the 15th March, 1955, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

[1] A. G. Pugsley, "Concepts of Safety in Structural Engineering." J. Instn Civ. Engrs, vol. 36, p. 5 (March 1951).

In the early days, as Pugsley remarks, most maximum external loads acting on bridges and buildings, apart from wind loads, were thought to be clearly definable and, in most cases, no question of the likelihood of their actual occurrence arose. In aeronautical engineering, however, ideas of the probability, or frequency of occurrence of loads had to be introduced into considerations of margins of safety. Laboratory tests to destruction of complete wings of an aircraft type which had shown some failures from overloading in service, permitted application of the results of actual loading experience to other cases. Such is not the case for large civil engineering structures where it is usually impossible to depend on tests to destruction and where external loads are frequently difficult to measure and are, to a certain extent, uncontrollable. Empirically limited working stresses, linked with conventional stress calculations for idealized loading cases, are therefore generally used.

On the other hand, where external loads can be controlled by the designer, a new outlook on margins of safety, and therefore, greater overall structural economy becomes possible. For no structure can the maximum load to which it will ever be subjected be forecast with absolute certainty. There is, therefore, always some " accident risk " involved, although the risk may be extremely small. The responsibility as to what extent this possibility of excessive loads, as well as to what extent deficiencies of design and workmanship, corrosion, etc., should be allowed for in the design, does not normally fall upon the designing engineer ; building codes or other regulations specify the design load and the greatest allowable stresses with which he may design permanent structures. Such specifications, however, are general rules and must necessarily be rather conservative. Correspondingly, inaccuracies of design assumptions are usually towards the safe side only.

There is often a regrettable tendency to regard extreme safety in a structure as a virtue in itself, without due consideration to whether such a degree of safety is actually necessary, or whether the public actually wants to be protected to such an extent. Where heavy load combinations approaching, or reaching, the design load are likely to occur only very rarely or not at all, it does not seem reasonable to consider the design load on the basis of the usual allowable stress (that is, to provide the full factor of safety) especially in cases where the " load factor of safety " probably considerably exceeds the " stress factor of safety." The fact that extreme load combinations, such as the coincidence of an extreme wind with an extreme snow load on buildings, are not very likely has been recognized in practice, and is often allowed for by an increase in permissible working stresses. Another example of the improbability of coincidence of extreme loads is in the design load for tall buildings since it is assumed that it is unlikely that on all floors the maximum floor loading will occur simultaneously.

For temporary structures, an attempt can be made to determine the frequency or probability of occurrence of heavy load combinations. This generally means making a statistical study of continuous records of traffic loads passing over the structure. If continuous records of strains occurring in a main member are taken, these would indirectly represent traffic loads. They can, however, for practical reasons, be considered in terms of stresses, rather than strains, or loads. If the results are plotted, a typical frequency-distribution diagram would generally be obtained. In such a diagram, stresses corresponding to loads of the magnitude of the design load would probably be beyond the values that can be shown by such a curve. Consequently, in all practical cases where records cannot be obtained over extremely long periods, the frequency of very heavy loads may only be obtained by extrapolation of the curve. Extrapolating from frequencies of measured loads to the frequencies of extreme loads is difficult, however, and sometimes of doubtful value. When certain definite loads become predominant, such as the weights of street-cars, and ready-mix concrete trucks, the shape of the frequency-distribution curve varies greatly from the normal curve, which is obtained for instance for rain storm precipitation, wind pressures, etc., and statistical rules cannot be applied. The theory of probability, then, is no substitute for the measurements of a great number of actual loads, because no matter how much time is spent on curve fitting, the accuracy of the curve for extreme loads is uncertain. Consequently, sound judgement has to be used without much quantitative help from statistics. The magnitude of the extreme loads, however, can often be fairly accurately known because of the truck size distinctions.

The Toronto Subway

The foregoing considerations were, in part, responsible for the decision of the Division of Building Research of the National Research Council of Canada to undertake a study of the strains actually developed in the steel deck beams of a large temporary structure. This was the temporary roadway which was necessary in the down-town business district of Toronto, capital city of the Province of Ontario, Canada, to permit the construction of a modern underground rapid transit line.

This project, which will cost about sixty million dollars, is being undertaken by the Toronto Transportation Commission, a public body responsible for all public transport in the city. The National Research Council's Building Research organization was privileged to use the subway project, during construction, as a large-scale research laboratory. The Author spent two years as research engineer on the project, working co-operatively with the construction staff of the Toronto Transportation Commission. The Commission, in addition to assisting the National Research Council staff mainly through the personnel of its Testing Subsection, also made available the following:

(1) Auxiliary equipment, such as the special hut for housing the equipment during the tests.
(2) The use of the soils laboratory, and of some of the workshop services.
(3) The use of the crane car with crew for loading of the deck beams.
(4) The use of the automobile assigned to the Testing Subsection.
(5) The co-operation of the traffic inspectors during the loading of the deck beams.

The National Research Council supplied personnel and all scientific equipment.

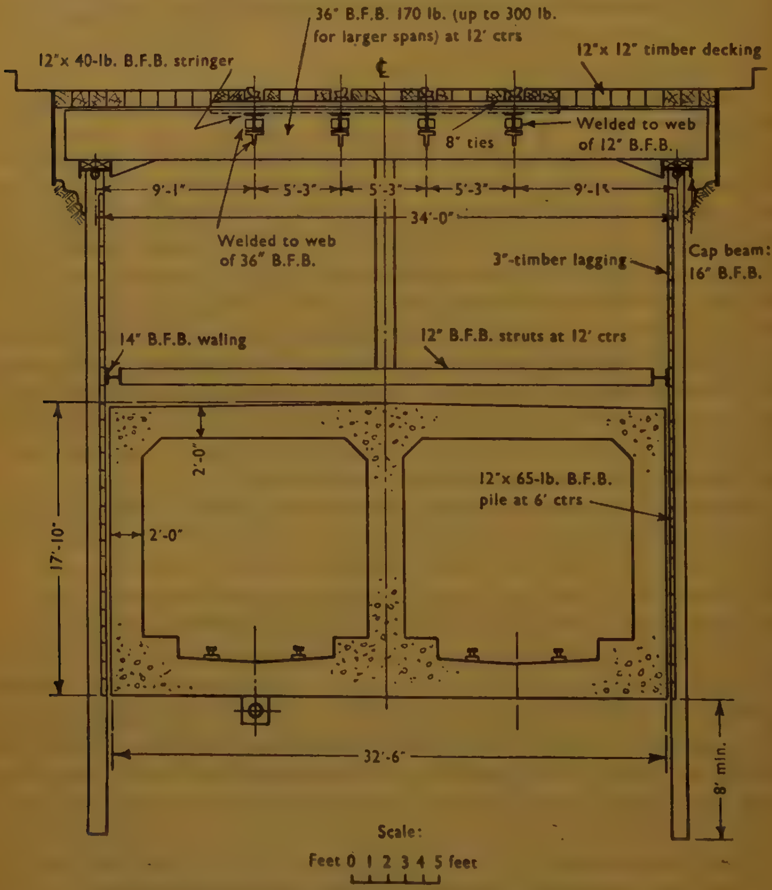
The first line of the proposed Rapid Transit system is the Yonge Street Line, now often referred to simply as the Toronto Subway. Of the total length of 4·6 miles, roughly one-third has been constructed under heavily travelled streets in the heart of the city, mainly under Yonge Street and Front Street, by the cut-and-cover method of construction. This method, by which the subway is built from the ground surface, was imperative because the subway was designed to be as shallow as possible to facilitate passenger transfer to and from surface transport, and because of the local geological formation. During construction, all normal traffic including street-cars was carried on a temporary road deck, while the major part of the excavation and the construction of the reinforced-concrete subway structure proceeded underneath.

Support of the sides of the cut against earth pressure, and support for the loads on the deck was achieved by steel H-piles, known as soldier piles, which were driven into the ground along both sides of the street, at 6-to-8-foot intervals. The first part of the excavation was then carried out, to take care of the great number of utilities, and to install the road deck. Normal street traffic was resumed on the temporary road deck, while construction work continued underneath. Wooden lagging was inserted between the soldier piles as the excavation deepened until it reached final grade, which is about 30 to 40 feet below the street surface. The actual subway structure, consisting of a reinforced-concrete " box," was then constructed, sand backfill placed on top, and the street repaved after removal of the deck.

The remaining two-thirds of the subway consisted partly of cut-and-cover sections not under city streets but on a private right of way, a little distance from Yonge Street, and partly of open cut, with cross-streets on overhead bridges.

Various types of road deck construction were used, depending on the required span, the available clearance underground, and the steel sections available at the time of construction which was a period of steel shortages. For the box sections of the subway (that is, between stations), intermediate sections (transitions, etc.), and some of the narrower stations, 36-inch broad-flange beams of various weights were used; for station sections either trusses or 36-inch broad-flange beams with knee braces or posts to reduce the unsupported span. Typical deck constructions of the three main classes are shown in *Figs 1* to *3*. Trusses, used extensively in the first station sections at Queen Street and at Union Station, were not used so

Fig. 1

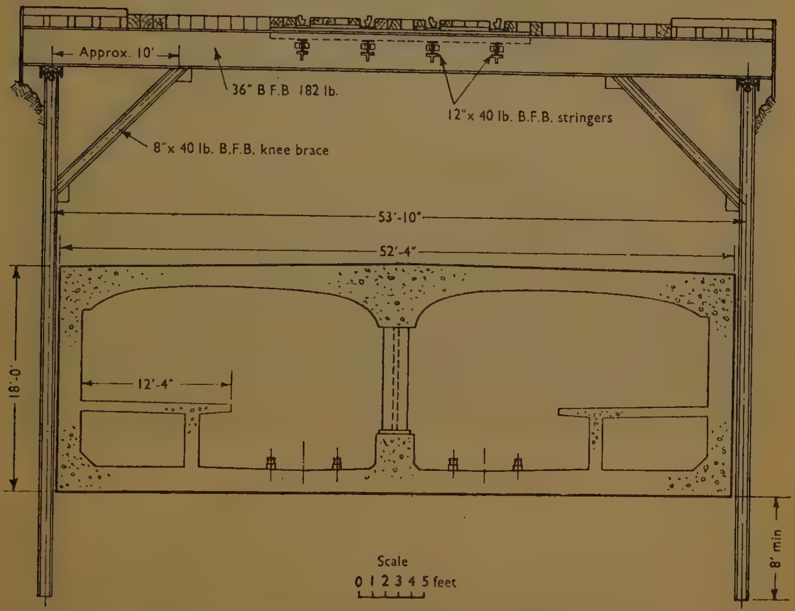

36" B.F.B. 170 lb. (up to 300 lb. for larger spans) at 12' ctrs
12"x 40-lb. B.F.B. stringer
12"x 12" timber decking
℄
Welded to web of 12" B.F.B.
8" ties
9'-1"
5'-3"
5'-3"
5'-3"
9'-1"
34'-0"
Welded to web of 36" B.F.B.
3"-timber lagging
Cap beam: 16" B.F.B.
14" B.F.B. waling
12" B.F.B. struts at 12' ctrs
2'-0"
2'-0"
12"x 65-lb. B.F.B. pile at 6' ctrs
17'-10"
32'-6"
8' min.
Scale:
Feet 0 1 2 3 4 5 feet

Typical Beam Construction for Box Section of Subway

much later mainly because, although lighter in weight, they involved far more labour for placing and welding and were not so adaptable for re-use. Attention to the measurements was therefore concentrated on the beam-type deck construction.

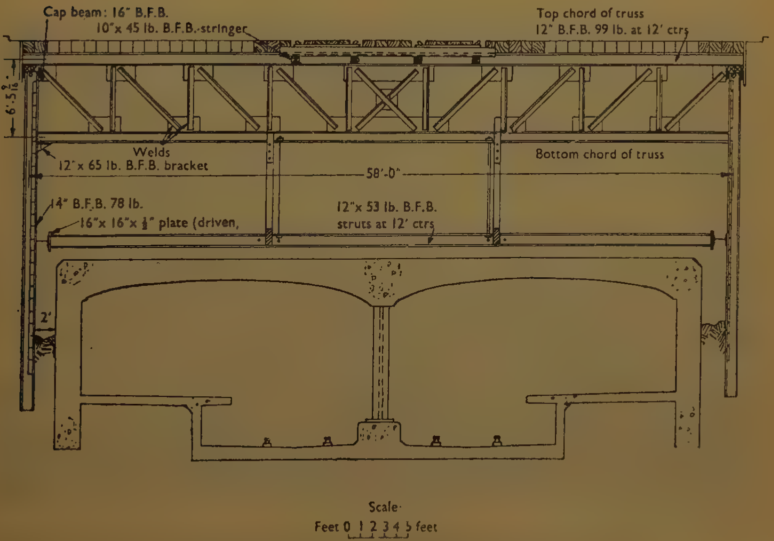
Construction of the road deck proper followed this sequence. First, along each side of the street, soldier piles were driven into the ground.

Fig. 2



Example of Beam with Knee Braces. Earth Sub-grade

Fig. 3



Example of Truss. Rock Sub-grade

The required depth of driving was 8 feet below the final grade level in soil, whilst in rock the piles were driven to refusal. The pavement and the street-car rails were then removed and the first lift of the excavation taken out from the surface by a power shovel. Excavation around the many utilities, however, had to be done by hand. The soldier piles were all cut off at the same depth below street surface and capped by steel beams. By means of two truck-mounted cranes, the main deck beams were then laid across the street at 12-foot intervals (*Fig. 4*) supported laterally by timber spreaders and tie-rods, and fastened to the cap beams by welded brackets. Stringer beams were next welded to the web of the main beams and 12-by-12-inch timbers laid along the outer lanes for use by the truck-mounted crane. The utilities were attached to the underside of the deck by steel cables or suitable timber construction (*Fig. 5*). With the laying of ties, street-car rails, and the remaining part of the deck timbers, the temporary roadway was completed and traffic was resumed (*Fig. 6*).

Strain Measurements

The majority of strain measurements were carried out on deck beams, since they were the elements of the temporary structure for which an improvement in economy of design would obviously be most important. Only a few measurements were made on one of the trusses used for the wider span of the station sections of the subway. Because of the much greater amount of labour involved in the cutting, assembling, and welding, trusses were abandoned in favour of broad-flange beams with knee braces.

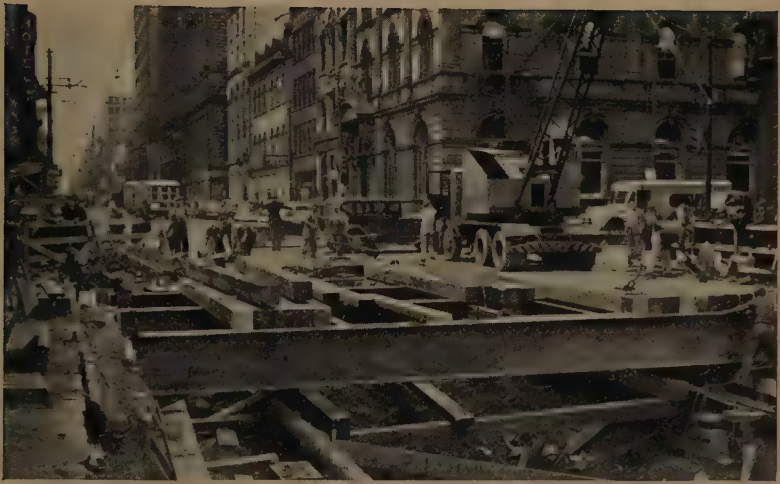
The knee braces not only served to reduce the free span of the wide flange beams, but also acted as partial shoring of the supporting piles against earth pressure. They, therefore, represented structurally one step in the transition from a frame to an arch with a corresponding reduction of bending moment. Appreciable stresses were accordingly expected in the knee braces themselves, especially in view of the fairly light steel sections used; a number of strain measurements were consequently made on these braces.

The stringers represented the main longitudinal element in the deck which could be considered as a grid system for analysis. Consequently, when stresses in the deck beams were found to be low, measurements were also made on some stringers to investigate the distribution of load from one loaded deck beam to its neighbours.

Since all deck beams had constant cross-sections, measurement in the section of the maximum stresses was considered sufficient. Strain gauges, therefore, were attached at the centre of the span, which in most cases coincided approximately with the centre-line of the street and of the street-car tracks. In one case, measurements were also taken at the quarter-points of the span.

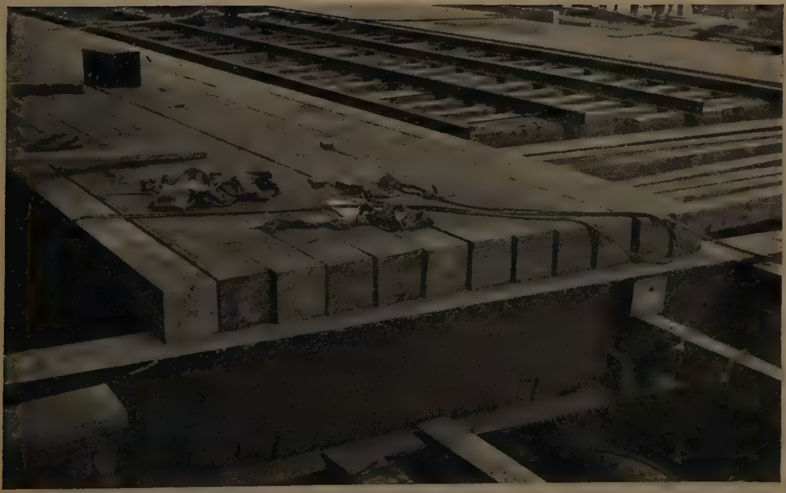
In accordance with the already stated objectives, of determining both

Fig. 4



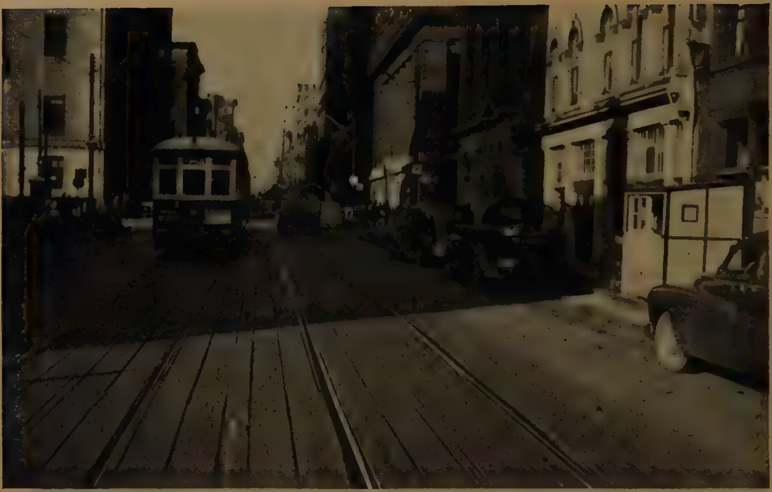
Placing of 36-inch Broad-Flange Beams on Capped Soldier Piles and 12-inch-by-12-inch Timber Planking

Fig. 5



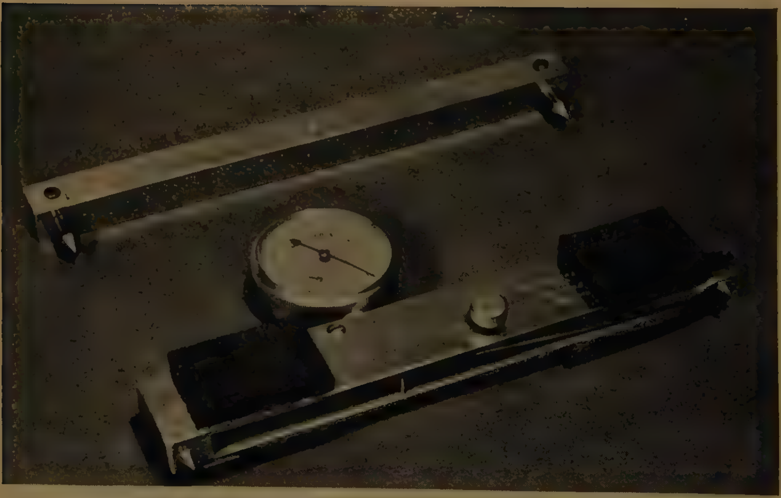
Timber Planking Cut to Length. Placing of Ties and Rails. Note the Suspended Water Main

Fig. 6



Temporary Road Deck Completed. Note Instrument Hut for Strain Measurements on Right

Fig. 7



Eight-inch Extensometer

the magnitude and the frequency of traffic load stresses, the two main types of strain measurements made were :—

(1) Recording of strains in deck beams caused by very heavy loads (street-car, weighed Toronto Transportation Commission crane-car, and weighed ready-mix concrete truck).
(2) Recording of strains caused by normal traffic (street-car and motor traffic) over longer periods of time.

The following two special strain tests, on one beam each, were also made :—

(*a*) Determination of impact effect on strain in deck beam from passage of slow and fast moving street-cars over rail crossings.
(*b*) Determination of effectiveness of knee braces in reducing strain in the main deck beams.

and in addition, there were two special investigations :—

(*c*) Determination of the continuity effect in the stringers.
(*d*) Observation of the simultaneity of heavy loads (street cars and ready-mix concrete trucks) on one beam during a concrete placing operation.

After considerable study, it was decided to take strain measurements with two types of gauge, an 8-inch mechanical extensometer (*Fig. 7*), and electrical resistance-wire strain gauges, known as SR-4 gauges, together with amplifiers and a direct writing recorder.

Experimental Results

In order to determine the maximum stresses in the deck beams, a combination of loads corresponding closely to the assumed design load was used. It was found, however, that owing to traffic on Yonge Street, it would be too difficult to position the combination of cars used (consisting of a crane-car, a street-car of the Toronto Transportation Commission, and two or more heavy trucks), all at one time on one beam. Strains caused by these loads acting separately were, therefore, measured and added together, using the law of superposition, except for the crane-car and the street-car the load effects of which were measured simultaneously. The law of superposition may not be strictly valid for the deck because the continuity effect of the deck timbers may contribute to a progressively increasing distribution of the load on to the adjacent beams. Should this be the case, however, the resulting sum of strains (or stresses) produced by the separate loads could only be greater than the strain (or stress) produced by the combined load, and therefore the deck beams in reality would be safer than would appear from the test results.

According to the specifications of the Toronto Transportation Commission, the design load for the temporary street deck consisted of the following loads :—

(1) A train on each street-car track, consisting of two or more 50-ton double-bogie cars, 40 feet long ; each axle load to be 12·5 tons ; axle spacings to be 5, 20, and 5 feet.

(2) A column of trucks on each traffic lane of 10-foot width, consisting of one 20-ton truck, preceded and followed by 15-ton trucks ; the load on the rear axle (or axles) to be $\frac{4}{5}$ of the total truck load and the spacing of the trucks 44 feet.

Considering one deck beam alone in a simplified way, the design load consists of the following double axle loads acting mainly on one beam :—

1 truck	1 street-car	1 street-car	1 truck
16 tons	25 tons	25 tons	16 tons

The load used in the experiments, although varying slightly from test to test, was generally as follows :—

1 truck	1 crane-car	1 street-car	1 truck
17 tons	33 tons	17½ tons	17 tons

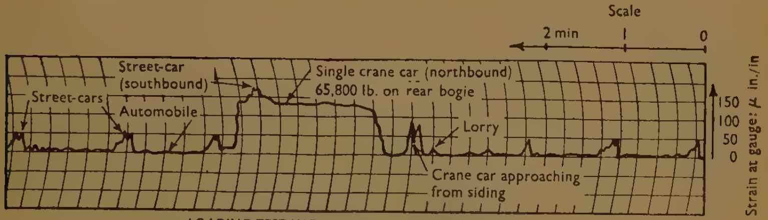
It can be seen that the magnitude of the test load with regard to its effect on bending moments was reasonably similar to the design load.

The strains and resulting stresses measured in the various beams under the above-mentioned loads are given in Table 1 (Plate 1), in which the following load cases are shown separately : one street-car north-bound ; one street-car south-bound ; crane-car north-bound ; crane-car north-bound plus street-car south-bound ; one concrete truck north-bound ; one concrete truck south-bound ; the last three loads combined. The maximum stresses obtained range from 5,500 lb. per square inch to 8,500 lb. per square inch. *Fig. 8* gives a reproduction of two test records obtained during load tests.

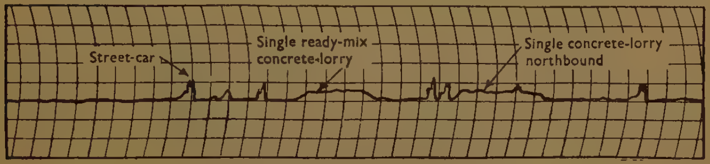
For each beam, investigated strains caused by normal traffic on Yonge Street were recorded for various periods of time, to obtain an idea of the frequency of occurrence of various loads. For the beam at Shuter Street a continuous twenty-four-hour record of strains caused by all traffic was obtained. *Fig. 9* shows distribution of strains (grouped by the numbers of lines of chart deflexion) during the twenty-four-hour period. *Fig. 10* gives the same measurements in a different manner ; for typical hours of the day the number of occurrences of the various strains is plotted. In *Fig. 16* the characteristic two pairs of strain peaks caused by the bogies of the front car and the trailer car can be seen.

The effect of impact is usually greatest at street intersections, where street-cars passing over the rail crossings cause vibrations of the deck. This effect was determined in one case (for a beam at the intersection of Yonge Street and Wellington Street) by comparison of strains caused by

Fig. 8



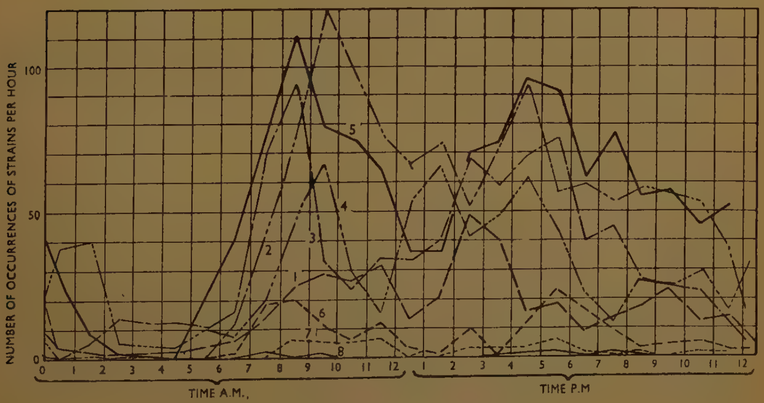
LOADING TEST WITH WEIGHED T.T.C. CRANE CAR WESTBOUND PLUS STREET CAR SOUTHBOUND (PASSING WHILE CRANE CAR STANDS IN CORRECT POSITION)



LOADING TEST WITH WEIGHED READY-MIX CONCRETE-LORRY ON TRAFFIC LANES, BOTH NORTH AND SOUTHBOUND

PART OF STRAIN ANALYSER RECORDS OF LOADING TESTS ON BEAM AT SHUTER STREET

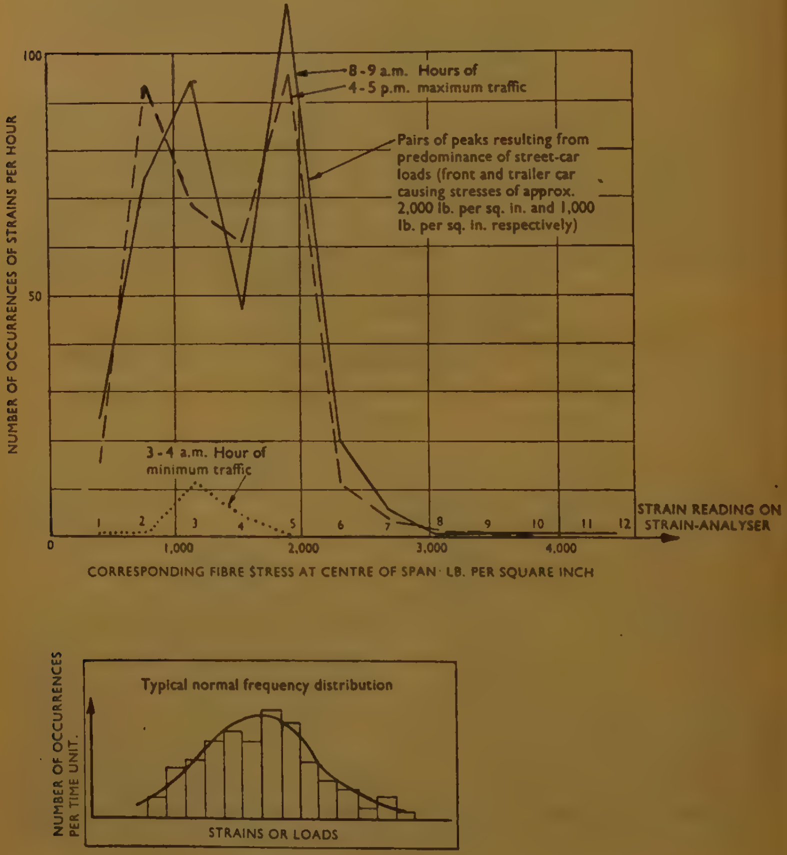
Fig. 9



Legend	Strain readings: lines	Fibre stress at centre of span
——— ——— ———	1	190–580 lb. per sq. in.
———·· ———	2	580–960 " " " "
———·———	3	960–1,350 " " " "
—— — — ——	4	1,350–1,730 " " " "
———————	5	1,730–2,120 " " " "
— — — — —	6	2,120–2,510 " " " "
-·-·-·-·-·-	7	2,510–2,890 " " " "
———————	8	2,890–3,280 " " " "

GRAPH OF FREQUENCY OF TRAFFIC LOADS (Beam at Shuter Street) a street-car crossing the intersection at full, and at very low, speeds. The increase of strain from full speed operation compared with the strain from static load was for this beam about 30 per cent for a single street-car. The impact effect of multiple loads was not determined. It was observed,

Fig. 10



Typical Frequency Distribution of Traffic Loads

(Obtained from strain measurement records taken during 24-hour period 28 Feb./1 Mar., 1951. Beam at Shuter Street.)

however, that the heavier the load combination became the more of its parts were moving slowly or even stationary. Consequently, the impact effect decreased with increasing load.

Effectiveness of Knee Braces

Owing to the fact that it was possible to test one station section beam (north of Wellington Street) in three different conditions—with knee braces, with one only, and with none—under normal traffic, an approximate idea of the effectiveness of knee braces in reducing the centre-span stresses was obtained. It was not possible, however, to load the beam with the heavy load combination for all three conditions. Table 2 (Plate 2) shows the stresses as determined from the strains measured for the loads of one street-car north-bound and one street-car south-bound for the above mentioned three conditions. The difference between the strains caused by the north- and south-bound street-cars arises because the strain gauges could not be mounted exactly in the centre of the span, owing to a spreader beam. The gauge was 10 inches east of the centre of the street as defined by the street-car rails.

The east knee brace was removed first as part of construction operations. The west knee brace was also removed while traffic on Yonge Street was still maintained. It was cut only near its lower end by an acetylene burner in such a manner as to become effective under heavier loads. The gap created by the burner was just wide enough so that it would not be closed under deflexion from the load of one street-car.

The increase of stress from removal of one knee brace was, on the average, almost 100 per cent; removal of both knee braces was approximately 200 per cent. If this same increase is applied to the stress from the maximum load combination (6,200 lb. per square inch obtained by superposition) a maximum fibre stress of almost 19,000 lb. per square inch would have been reached.

The stresses in one of the knee braces of the above mentioned beam were also measured. The conclusion that the knee brace would experience the highest stresses near mid-span on the underside was confirmed by taking spot readings at various points of the knee brace by means of the 8-inch mechanical extensometer. Electrical strain gauges were then attached only at mid-span of the brace and the results shown in Table 2 were attained by the gauge on the underside.

The maximum stress in the knee brace, caused by the usual heavy load combination totalling 84½ tons, was 8,550 lb. per square inch compared to 6,200 lb. per square inch in the deck beam. Both these figures were obtained by superposition. Consequently, the knee brace was under-designed compared to the beam, in fact, considerably more than appears from comparison of the stress values, because of the danger of buckling present in the knee brace.

Continuity Effect of the Stringers

The low values of maximum stress obtained in the deck beams led to the conclusion that these beams received considerable load relief, compared to strict simple beam action, by distribution of load to varying degrees to the adjacent beams through stringers, rails, and timbers. This load relief was caused by the greater deflexion of the loaded beam, except when all beams were equally loaded (which was never the case with the spacing of beams and loads used on this project). The partial fixity of the ends of the deck beams (welded brackets) also contributed to the load relief, although only to a minor degree.

The stringers acted partly as continuous beams through the webs of the deck beams to which they were welded by brackets. The transmission of bending moments through the web of the heavy deck beams was, of course, impeded by the torsional stiffness of the deck beams. As can be seen from the strain lines of Figs 11, Plate 2, however, some negative moments did act in the non-loaded spans adjacent to the loaded span. The strain readings were taken by means of an 8-inch mechanical extensometer on two lines of stringers over two spans, at the quarter- and mid-span points. The load used consisted of the Toronto Transportation Commission crane-car loaded to approximately 32 tons per bogie, that is, exceeding the specified design load by 7 tons. For the measurements the deck planks covering the ties were removed and the engineer reading the extensometer was standing just below the deck on the partly completed backfill.

The maximum stress recorded in the stringers was 9,400 lb. per square inch, 25 per cent below the value of 12,600 lb. per square inch computed for the same steel section acting as a simple beam. The greatest stress measured in an area of negative moment was approximately 1,000 lb. per square inch.

From the magnitude of strains in the stringers for the load straddling a deck beam, it can be seen that a considerable portion of the load of a truck, or a street-car, or crane-car is always distributed to the adjacent beam.

Coincidence of Heavy Loads During Concrete Placing

The likelihood of occurrence of heavy load combinations was greatest during concrete placing operations, when in addition to two street-cars crossing on one beam there might be two ready-mix concrete trucks standing on the side lanes on the same beam. It is not surprising, however, that during the accurate observation made of one complete placing operation for a roof section, approximately 100 feet south of Dundas Street, during about five hours, not once was a beam loaded simultaneously by two street-cars and two concrete trucks. This becomes understandable when it is considered that only once every two to two and a half hours do two street-cars cross on the same beam, doubling the strain caused by one street-car. This was established from the records obtained by means of the strain analyser equipment.

During the above mentioned observation of load combinations occurring during a concrete placing operation the following frequencies of loads were found :—

Two street-cars and two concrete trucks exactly on the same beam : 0 times.

One street-car and two concrete trucks exactly on the same beam : 0 times.

One street-car and two concrete trucks with centre of rear axles at average distance of 1 foot from the beam : 0 times.

One street-car and two concrete trucks at average distance of 2 feet : once.

One street-car and two concrete trucks at average distance of 3 feet : twice.

One street-car and 2 concrete trucks at average distance of 4 feet : 8 times.

One street-car and two concrete trucks at average distance of 5 feet : 7 times.

It follows that even during concrete placing operations, load combinations consisting of two street-cars and two concrete trucks acting simultaneously on the same beam are extremely rare, although definitely possible.

Discussion of Results

Low Maximum Stresses

The two most striking aspects of the results are the low value of stresses obtained under a heavy load combination corresponding closely to the design load and the infrequency of heavy loading by the traffic on the deck. It is believed that the low stresses in the deck beams principally resulted from the fact that they did not act as simple beams. Two reasons for this were :—

(1) Owing to the load relief from the continuity effect of stringers, rails, and deck timbers, the maximum load actually carried by one beam does not correspond to the combined axle loads acting on this beam.

(2) The welded connexions between the deck beams, the cap beams, and piles introduce a small measure of end fixity into the beams.

The amount of axial load from earth pressure that is transmitted from the soldier piles to the deck beams is limited by the conditions present. The earth pressure acts mainly as a horizontal load on the piles, which are supported at the top by the deck beams, at the bottom by the embedment in the soil and, in some cases, medially by struts. The flexural strength of the steel piles, combined with their unsupported vertical span below the deck beam, limits the reaction in the deck beam and, therefore, the stresses from earth pressure. An estimate for an average deck beam, with struts 10 feet below it, shows an average compressive stress of less than 3,000 lb. per square inch even if the piles were loaded to yield (40,000 lb. per square inch fibre stress). Actually, this force is introduced along the bottom of the deck beam, resulting in a favourable eccentric loading, that is, a tendency to compensate the stresses due to the deck load. In knee-braced deck beams the axial load can become greater because the knee braces, in assuming part of the role of the struts, transfer a greater percentage of the earth pressure to the deck beam.

Infrequency of Heavy Loads

As the records of strains in the deck beams caused by normal traffic show, the other notable aspect of the results is the infrequency of large strains or stresses. It is probable that the majority of the deck beams on the project never normally experienced stresses exceeding three-quarters of the stress obtained under the heavy experimental load combination used, and were subjected to this stress only very seldom, that is, only when the crane car was used on Yonge Street. In the absence of the crane car, the probability of relatively heavy loads acting simultaneously on the same beam was greatest during concrete placing operations. Even then, however, very few beams experienced the full combined load of two street-cars and two full concrete trucks, as the observation of vehicle movements made during a concrete placing operation showed.

Improvement of Economy in Future Designs

One of the principal objectives of this study was to see in what direction improvements might be affected in future designs of the same type of temporary deck structure. The design loads used for this part of the subway were necessarily used in the absence of such specific information as this Paper presents. Nothing in this Paper is to be taken as any indication of criticism of this quite proper approach to an unusual design problem. Should, however, a similar temporary structure have to be designed in the future, in Toronto or elsewhere, it does now seem possible for some additional economy to be introduced safely into the proportioning of the deck system.

First, it seems practicable to make some allowance or reduction in the assumed design load on any one beam, for the continuity which obviously exists even in such a temporary structure as that which was investigated in Toronto.

To allow for the infrequency of heavy load combinations a further decrease of the design load would seem possible, because the probability of occurrence of the heavy load combinations is small. In fact, a considerable saving might be obtained by regulating the movement of such vehicles as the crane-car, by issuing instructions with regard to passing other vehicles on any temporary deck. If, in this way, drivers were not

STRAIN MEASUREMENTS ON THE TEMPORARY ROAD DECK FOR THE TORONTO SUBWAY

PLATE 1

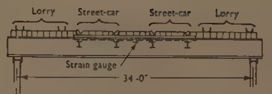
TABLE 1.—STRESSES IN DECK BEAMS

Subway section	Box section	Box section	Box section	Station section	Station section
Location	At Shuter Street	At Wellington Street	At Dundas Square	North of Wellington Street	North of Queen Street
Chainage	136 + 35	114 + 35	144 + 00	115 + 44	132 + 60
Type	I-beam	I-beam	I-beam	I-beam knee-braced	Truss
Steel section	36″ B.F.B. 182 lb.	36″ B.F.B. 230 lb.	36″ B.F.B. 182 lb.	36″ B.F.B. 182 lb.	Chords: 12″ B.F.B., 120 lb. and 45 lb.
Span	34 feet	37 feet 10 inches	34 feet		54 feet
Spacing of beams	12 feet	South: 6′; north: 12′	12 feet	12 feet	12 feet
Date of test	1 and 2 March, 1951	12 and 18 Jan., 1951	17 July and 24 Aug., 1951	12 Dec., 1950	17 July and 27 Aug., 1951
Gauge to extreme fibre	4″ from top	4″ from top	4″ from top	4″ from top	Centre of bottom chord
Gauge to centre-line of beam	2″ (West)	2″ (West)		10″ (East)	6′ 6″ (East)
Centre-line of road to centre-line of beam	2″ (West)	11″ (West)		5″ (West)	

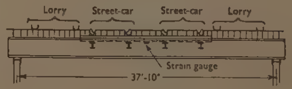
For all measurements: Type of strain gauge SR–4, A.1. Gauge factor 2·05.

Instrument attenuation:—Strain: 1 line on chart = 10μ in./in.
Stress: 1 line on chart = 300 lb./sq. in.

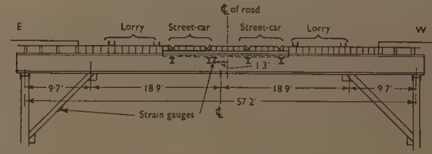
Loads on deck beam:	Strain (lines)	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain (lines)	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain (lines)	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain (lines)	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain (lines)	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.
One street-car (northbound) (loaded, 17½ tons bogie)	5½	1,650	2,100	3½	1,050	1,350	4½	1,350	1,750	3	900	1,150	4½	1,350	1,350
One street-car (southbound) (loaded, 17½ tons/bogie)	5½	1,650	2,100	3	900	1,050	5	1,500	1,950	2	600	750	3	900	900
One crane-car (northbound) (loaded, 33 tons/bogie)	13	3,900	5,000	8½	2,550	3,300	13	3,900	5,000	10½	3,150	4,050	12	3,600	3,600
Crane car (northbound) + street care (southbound) (loaded 50½ tons)	17½	5,250	6,750	10½	3,150	4,050	—	—	—	12	3,600	4,650	—	—	—
Concrete-lorry (northbound) (on side lane. 17 tons)	2	600	775	2¼	675	850	2	600	750	3	900	1,150	3 (assumed)	—	—
Concrete-lorry (southbound) (on side lane. 17 tons)	2	600	775	1½	450	600	—	—	—	1	300	400	2 (assumed)	—	—
Maximum load (total of last three: 84½ tons by superposition)	21½	6,450	*8,300*	14¼	4,275	*5,500*	22	6,600	*8,500*	16	4,800	*6,200*	20	6,000	*6,000*
One slow street-car (northbound) (for minimum impact)				2¾	825	1,050									
One slow street-car (southbound (for minimum impact)				2¼	675	850									



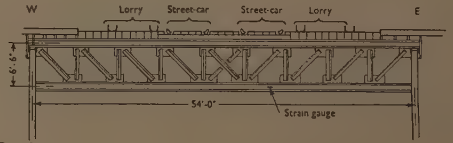
Shuter St and Dundas Square



Wellington St



North of Wellington St



North of Queen St

PLATE 2

STRAIN MEASUREMENTS ON THE TEMPORARY ROAD DECK FOR THE TORONTO

TABLE 2.—STRESSES IN KNEE-BRACED DECK BEAM

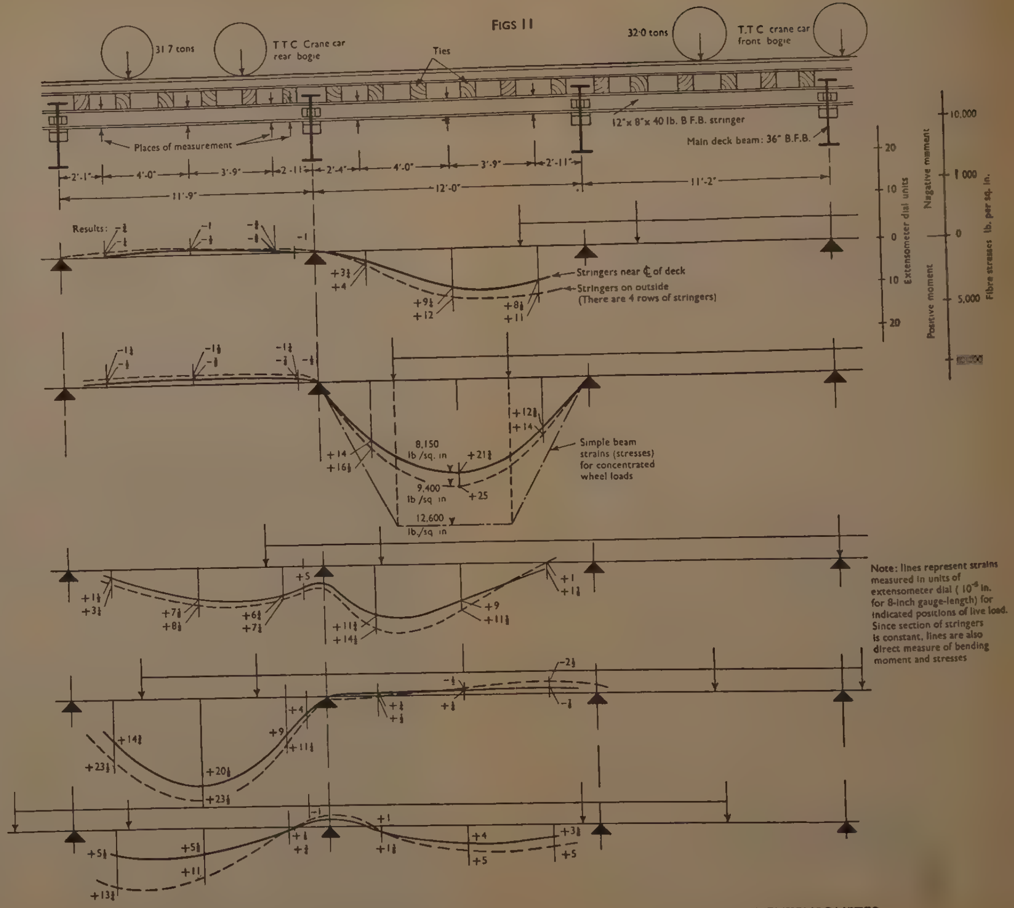
Subway section . . .	Station section
Location	North of Wellington Street
Chainage	115 + 44
Type	I-beam, knee-braced
Steel section . . .	36″ × 182 lb. B.F.B. Knee braces: 8″ × 40 lb. B.F.B.
Span	Total: 57′ 2″. Between knee braces: 37′ 10″
Spacing of deck beams .	12 feet

	Strains and stresses in:											
	Deck beam with both knee braces						Deck beam with only one brace (E brace removed)			Deck beam with both braces removed		
	Deck beam			Knee brace								
	Strain lines *	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain lines	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain lines	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.	Strain lines	Stress at: gauge lb./sq. in.	Stress at: extreme fibre lb./sq. in.
One street-car—north (loaded 17½ tons per bogie)	3	900	*1,150*	5	1,500	1,500	5½	1,650	*2,100*	8	2,400	*3,100*
One street-car—south (loaded 17½ tons per bogie)	2	600	*750*	3½	1,050	1,050	4	1,200	*1,500*	6½	1,950	*2,500*
One crane-car—north (loaded 33 tons per bogie)	10½	3,150	4,050	14	4,200	4,200			83% † and			167% † and
One crane-car—north + one street car (loaded 50½ tons)	12	3,600	4,650	15½	4,650	4,650			100% †			225% †
One concrete-lorry—north (on side lane, 17 tons)	3	900	1,150	9	2,700	2,700						
One concrete-lorry—south (on side lane, 17 tons)	1	300	400	4	1,200	1,200						
Max. load of last three, by superposition, 84½ tons	16	4,800	6,200	28½	8,550	8,550						

* See Table 1.

† Increase of stresses compared to beam with both knee braces.

FIGS 11



Note: lines represent strains measured in units of extensometer dial (10^{-4} in. for 8-inch gauge-length) for indicated positions of live load. Since section of stringers is constant, lines are also direct measure of bending moment and stresses

STRAIN MEASUREMENTS ON STRINGERS ("TRACK BEAMS") BY MEANS OF AN 8-INCH EXTENSOMETER allowed to pass a street-car and two heavy trucks at the same time, consideration of a further reduction in the design load would be possible. Even if drivers of specially heavy cars neglected to observe instructions, the factor of safety inherent in the allowable design stress would be ample to take care of the slight temporary overloading.

The investigation of the effectiveness of the knee braces showed the great benefit produced by these braces in reducing the bending moments in the deck beams. The knee braces would seem, therefore, to be of great value in saving steel in the deck beams, and possibly also in the box sections of the subway, when circumstances allow their use. The knee braces help to reduce the need for strutting by transferring a greater percentage of the earth pressure from the piles to the deck beams.

ACKNOWLEDGEMENTS

Much assistance has been received by the Author during his work. It would be impossible to list all those who have given a helpful hand in the work itself or in the form of suggestions during discussions. Special mention, however, must be made of the co-operation of the Toronto Transportation Commission and the staff of the Testing Subsection of the Rapid Transit Department in particular, without whose help the work could not have been carried out. The co-operation of the general contractor for the down-town section of the subway, Messrs Pitts, Johnson, Drake and Perini, is also acknowledged. Gratitude is expressed for the many helpful suggestions (and the loan of some equipment) from the staff of the Research Laboratories of the Hydro-Electric Power Commission of Ontario.

The Paper is a contribution from the Division of Building Research, National Research Council of Canada. As noted, all the work described was carried out co-operatively with the Toronto Transportation Commission, of which Mr W. H. Paterson is Chief Engineer. Mr D. H. MacDonald, in charge of the Testing Subsection of the Rapid Transit Department worked closely with the Author in this study. The Paper is presented with the approval of the Commission and of the Director of the Division, Mr R. F. Legget, M.I.C.E., who followed the work described with his personal attention.

The Paper is accompanied by four photographs and nine sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

CORRESPONDENCE

on two Papers published in Proceedings, Part I, March 1954

Paper No. 5950

" The Variability of Concrete and its Effect on Mix Design " †

by

Francis Robert Himsworth, B.Sc., Ph.D.

Correspondence

Mr L. Boyd Mercer, of Melbourne, Australia, was surprised to learn from the Paper that the current procedure in mix design was apparently to conduct and statistically examine tests for one mean compressive strength and then apply the conclusions to another " level ". There could be little support for such action or for any contention that the coefficient of variation would remain constant for a range of mean compressive strengths. That had been accepted, if not generally adopted, since the work of R. W. Crum and H. W. Leavitt.[10]

Whilst it would be very simple to have a single standard deviation which would cover all mean strengths for the operations of a particular plant, there did not yet appear to be sufficient data available to support such a proposal. Moreover, upon examination of the long list of causes for concrete strength variation,[11] it appeared improbable that the same standard deviation could be obtained for mixes varying from the higher to the lower water-cement ratios. For instance, the effect of " cement variation " might not be the same at different water-cement ratios, whilst constant errors in batching water and cement would not necessarily give a constant error in the effect on the water-cement ratio, and resulting compressive strength.

It was suggested that pending the compilation of more complete supporting data, the safest procedure for mix design was to determine the actual standard deviation for mixes giving the desired mean compressive strength for each specific set of operating conditions,[12] and that was of course the same as using the coefficient of variation.

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† Proc. Instn Civ. Engrs, Part I, vol. 3, p. 163 (March 1954).

[10] L. Boyd Mercer, "Concrete Quality Control—Fallacious Application of Statistical Analysis." *Commonwealth Engineer*, vol. 38, p. 182 (Dec. 1950).

[11] L. Boyd Mercer, " Concrete Strength Variations—60 Contributing Causes." *Building and Construction*, Melbourne, (28 March, 1950).

Mr P. J. F. Wright observed that as a result of the accumulation, during recent years, of information on the variability of concrete several authorities had come to realize that the standard deviation of the results of works cube tests was largely independent of the average strength and that the standard deviation was more appropriate than the coefficient of variation or percentage range when designing mixes. That view has been held at the Road Research Laboratory for several years and the standard deviation was currently used there in designing concrete mixes.

The second part of the Paper was a useful study of the various factors affecting concrete strength but, in view of the very limited information on which the analysis appeared to be based, the conclusions might require to be modified in the light of future work. In particular, Mr Wright could not agree with the implications of the Author's first conclusion relating to Part 2 of the Paper. That conclusion had been drawn from the results of one contract only and was quoted in terms of total variation, a quantity which the Author had decried in Part 1 of the Paper. Further, the conclusion implied that the total variation in concrete strength might be divided into 10 per cent from one cause and 90 per cent from another, and that, as the Author had explained, could not be done. If, instead, the variance was considered, the components were directly additive, as the Author had pointed out. The proportion of the variance arising from variation in cement quality then appeared as 32 per cent at 7 days and 19 per cent at 28 days.

Those figures, however, were obtained in a manner which was open to question, since ordinary methods of regression analysis were not satisfactory where both variables were subject to errors of comparable magnitude. In Appendix 2 (p. 197) equations were given for the best-fitting straight lines relating C to M. If those lines were drawn on diagrams in which C and M were plotted (*Figs 5*) it was evident that: (1) there was no well-defined relation between C and M and (2) the slope of the regression line was lower than would be obtained if a straight line were to be fitted by inspection. It was well known that a regression line was not a valid estimate of the true relation underlying the two variables unless one of the variables (the one considered as the independent variable) was known without error. If it was assumed that the mortar strengths were obtained without error the lines quoted in Appendix 2 were obtained. If it was assumed that the concrete strengths were determined without error the lines obtained were:

$$C = 1{\cdot}74M - 3{,}450 \text{ at 7 days}$$

$$\text{and } C = 2{\cdot}17M - 7{,}800 \text{ at 28 days.}$$

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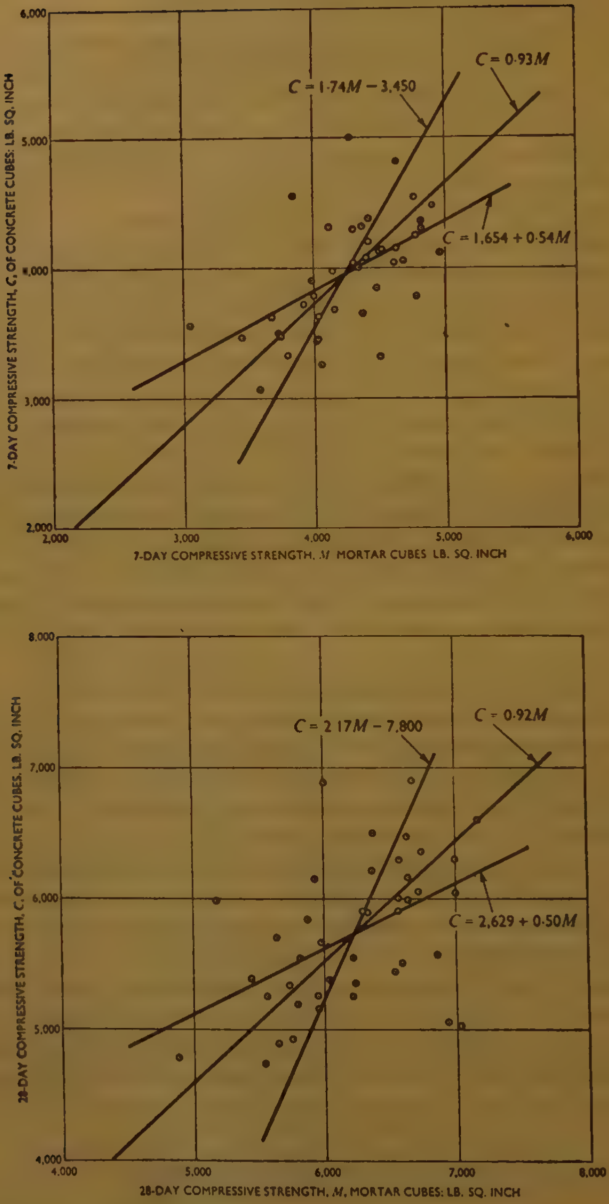
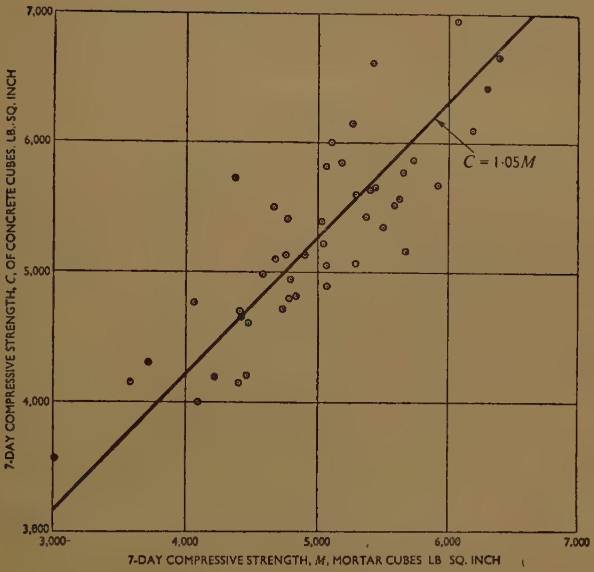
[12] L. Boyd Mercer, "Ready-Mixed Concrete: Quality Control Refinements." Symposium on Mix Design and Quality Control of Concrete, C.A.C.A., London, 1954.

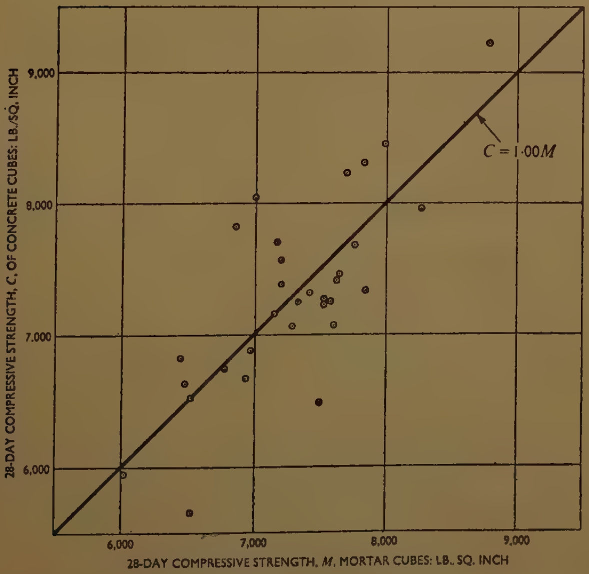
Figs 5



THE RELATION BETWEEN CONCRETE STRENGTH AND MORTAR STRENGTH—RESULTS USED BY THE AUTHOR

Figs 6





The Relation between Concrete Strength and Mortar Strength—Results obtained at the Road Research Laboratory

In reality the underlying relations, if any, probably lay between those quoted in Appendix 2 and those quoted above.

The effect of erroneously estimating the relation between C and M could be seen by examining the structure of the observed variations. It was reasonable to assume that:

$$M = M_i + \epsilon$$

$$\text{and } C = a + bM_i + \delta$$

where M_i denoted the true strength of any batch of cement (the values of M_i being distributed normally with variance $V(M_i)$) and ϵ and δ denoted random errors distributed normally about zero mean with variances $V(\epsilon)$ and $V(\delta)$. It followed that:

$$C - M = a + (b - 1)M_i + \delta - \epsilon$$

From those equations could be obtained:

$$V(M) = V(M_i) + V(\epsilon)$$

$$V(C) = b^2 V(M_i) + V(\delta)$$

$$V(C - M) = (b - 1)^2 V(M_i) + V(\delta) + V(\epsilon)$$

Hence

$$V(M_i) = \frac{1}{2b}\{V(M) + V(C) - V(C - M)\}$$

$$V(\epsilon) = V(M) - V(M_i)$$

$$V(\delta) = V(C) - b^2 V(M_i)$$

Thus the two components of $V(C)$ could be estimated if the true value of b was known. In practice the true value of b was not known and there was no satisfactory way of estimating it. Using for the value of b the regression coefficient found by minimizing the sum of the squares of the deviations in the direction of the C-axis, that was to say, the value obtained by the Author, gave $V(\epsilon) = 0$. Using for b the value obtained by considering deviations in the direction of the M-axis, gave $V(\delta) = 0$. Neither of those results was acceptable and, if an intermediate value of b was taken, the component of the variance of concrete tests due to cement variation was directly proportional to the value used for b.

The important problem was thus one of determining the true value of b. It could not be obtained by ordinary regression analysis, even if that was applied to a very much larger number of batches of cement. The true value of b had therefore to be estimated in some other way and one reasonable basis was to assume that the relation was a straight line passing through the origin and the point $(\bar{M}, \bar{C})$ where $\bar{M}$ and $\bar{C}$ denoted the average values of M and C respectively. That would give $b = \bar{C}/\bar{M}$ and if that value was used for b the following results were obtained:—

	7 days	28 days
Ratio C/M taken as b	0·93	0·92

Components of variance of mortar tests :—

Due to cement variation, $V(M_i)$	109,930	157,490
Random errors, $V(\epsilon)$	76,430	133,720
Total variance, $V(M)$	186,360	291,210

Components of variance of concrete tests :—

Due to cement variation, $b^2V(M_i)$	95,080	133,300
Random errors, $V(\delta)$	82,480	181,270
Total variance, $V(C)$	177,560	314,570

On that basis the proportion of the variance of the concrete tests which was due to variation in cement quality appeared as 54 per cent at 7 days and 42 per cent at 28 days. The same method of analysis applied to results obtained at the Road Research Laboratory over a period of years and recently analysed, gave corresponding values of 85 per cent at 7 days (48 batches) and 62 per cent at 28 days (31 batches). Those figures referred to concrete tests in which laboratory control had been exercised and the variation in cement quality would therefore be expected to provide the largest contribution to the variation. The results were plotted in *Figs 6* and showed a rather better relation than those in *Figs 5*. Further, those results which covered a wider range of strengths and showed less scatter helped to substantiate the hypothesis that the true relation was a straight line passing through the origin.

It was not claimed that that solution was necessarily correct and it was not considered possible at present to obtain an exact solution. The method put forward above, however, seemed more in accordance with experience than the Author's and would lead to the conclusion that about half the variance of the results of concrete tests obtained on a large well controlled job arose from variation in cement quality. The remainder was the combined effect of batching errors, sampling errors, and testing errors and thus cement variation might be considered to be the largest single factor.

Dr J. M. Plowman remarked that within the past year or two it had become fashionable to use statistical expressions such as "standard deviation," etc., to represent the variability of concrete made over a period. They were derived from crushing strengths of cubes, the number of which was usually small, except on large works such as aerodromes and roads. He had always understood that for statistics to have much meaning the number of samples should be large, as was possible in botany or biology. The contradictions arising in "public opinion polls" with samples of 50,000 or more were well known, yet one read of statistics being applied to concrete cubes numbering less than fifty.

He had as yet seen no mention, in relation to statistical analysis, of the error in reading the dial gauge of the crushing machine. All results were quoted as exact stresses and yet there was a wide range of possible error.

For example, in a series of tests using a 150-ton hydraulic press the oil-pressure gauge had been calibrated in divisions of 50 lb. per square inch and readings had been estimated to 25 lb. per square inch. If an error in reading, due to slight sticking of the pointer, parallax, or laxity of the operator, of 25 lb. per square inch occurred then Table 16 showed the possible error in calculating the stress in a 4-inch cube.

TABLE 16

Stress: lb. per sq. in. . .	3000	3500	4000	4500	5000	5500	6000	6500
% error	5·7	4·9	4·25	3·8	3·4	3·1	2·85	2·6
Stress: lb. per sq. in. . .	7000	7500	8000	8500	9000	9500	10000	
% error	2·45	2·3	2·1	2	1·9	1·8	1·7	

A set of fifty cubes with sand gradings constructed by recombining the sieve fractions $\frac{3}{16}$ inch–7, 7–14, etc., designed to test the variability of concrete fully compacted by vibration at a constant frequency and acceleration, had been divided into two groups by a rise in curing-room temperature. From those groups the following results had been obtained.

Group 1. 23 cubes: mean 7-day crushing strength 4,380 lb. per square inch; standard deviation 220 lb. per square inch.

Group 2. 27 cubes: mean 7-day crushing strength 5,075 lb. per square inch; standard deviation 224 lb. per square inch.

A 1 : 6 mix had been used, with a water/cement ratio of 0·45. The cubes had been wet cured.

From Table 16 the possible error from the pressure-gauge reading was 4·25 per cent at 4,000 lb. per square inch and 3·4 per cent at 5,000 lb. per square inch, or 170 lb. per square inch on a 4-inch cube. From the statistical analysis the results showed that there was a 1 : 1,000 chance of a cube having a strength greater or less than the mean by 174 lb. per square inch in the first case and 160 lb. per square inch in the second.

Dr Plowman suggested, therefore, that the error due to reading of the pressure gauge should be taken into account when considering statistics of cubes. At low strengths it might entirely dominate or mask the variation due to the concrete itself, as in the case quoted above. Furthermore, as in that case, many more results were necessary if any good was to be gained from the mathematical labour involved in such analyses.

Mr A. M. Neville, of Christchurch, New Zealand, observed that the variation in strength of concrete caused by a variation in the quantity of water in the mix might arise from the "incorrect" amount of water either present in the aggregate or gauged directly into the mixer. It seemed that the magnitude of that "error" was proportional to the quantity of water added to the mix, and it was unlikely to be greater the smaller the "correct" quantity involved.

Assuming such an error to be, say, 10 per cent of the correct amount of water, the variation in strength of concrete could be determined from strength-to-water/cement-ratio curves. Considering a high-strength mix of, say, 0·4 water/cement ratio and a low-strength mix of, say, 0·65 water/cement ratio, and referring to the Road Note No. 4 (Road Research Laboratory, 1947), the following decrease in strength would be found for a 10-per-cent increase in the water/cement ratio for 4-inch cubes of normal Portland cement, aged 28 days:

0·4 water/cement ratio	6,900 lb. per square inch
0·44 ,, ,, ,,	6,200 lb. per square inch
Decrease	700 lb. per square inch
0·65 water/cement ratio	3,600 lb. per square inch
0·715 ,, ,, ,,	3,000 lb. per square inch
Decrease	600 lb. per square inch

Similar results could be obtained for other water/cement ratios, and those results would tend to confirm the hypothesis that, within the limits of the usual mix proportions, the standard deviation was independent of the mean strength of concrete, whilst the coefficient of variation was higher the lower the mean strength. (In the case considered the coefficient of variation was 17 per cent at 3,600 lb. per square inch and 10 per cent at 6,900 lb. per square inch.)

If the "error" in water was assumed to be independent of the "correct" quantity, the above argument would be slightly modified but not invalidated.

Mr Neville offered the foregoing comments in the belief that the hypothesis propounded was borne out not only by a statistical study but also by an inspection of strength-to-water/cement-ratio curves.

The Author, in reply, observed that Mr Boyd Mercer, and probably other concrete engineers, did not act on the assumption that the coefficient of variation of concrete remained constant if the mean strength was altered; it was nevertheless true that many engineers who had become interested in mix design and quality control had, quite understandably, gained from the standard literature on the subject the impression that that was so. Mr Mercer's suggestions for a "safest procedure" were, of course, safe, but were of little assistance to an engineer involved in comparatively small usages of a number of quite different mixes, and did not help him to decide on the economic gain to be expected if close control was introduced.

It was true, as Mr Wright had pointed out, that a regression analysis could not be made if the two variables were both subject to errors of comparable magnitude without introducing some further assumptions, for example, postulating the value of b, or the ratio of the error variances. His assumption that $b = \bar{C}/\bar{M}$ might or might not be valid, but the matter required further investigation. The Author had, since writing the Paper, obtained some data which avoided the need for arbitrary assumptions. At one cement factory, a concrete cube had been made, in addition to the mortar cubes to B.S.12, as an additional check on cement quality. Over a period of a year, ninety-three such cubes had been made (1 : 2 : 4 : 0·6) and tested at 28 days. The mean strength had been 4,760 lb. per square inch and the standard deviation 273 lb. per square inch. The testing error for that laboratory was known to be about 150 lb. per square inch, so that, assuming no sources of variation other than cement quality and testing error, there was found for cement quality a standard deviation of 228 ($= \sqrt{(273^2 - 150^2)}$) lb. per square inch. That was for a single factory, but it was in good agreement with the figure in the Paper.

The general conclusion, however, was not affected, even taking the higher figure deduced by Mr Wright, 425 lb. per square inch. Complete elimination of cement-quality variation would then reduce the overall standard deviation of concrete strength from 561 to 427 lb. per square inch, that was, by 24 per cent. The mean strength could then be reduced by 2·33 (561–427) = 312 lb. per square inch, which would represent a negligible economy. Halving the cement variation, a more realistic assumption, would mean a decrease in mean strength of only 147 lb. per square inch. Those figures applied to conditions of careful batch control; with less careful control the savings would be less. That was the only logical way of assessing the value of a reduction in cement variation. Mr Wright's suggestion of comparing the variances attributable to the different sources of variation, though properly used in many applications of statistics, was inappropriate to the present purpose.

Another point which arose from Mr Wright's analysis was that the standard deviations of mortar and concrete strengths, after allowing for cement variation, were 366 and 425 lb. per square inch respectively. That for mortar strength should include no appreciable contribution for variation in water/cement ratio, and was surprisingly large. It might have been expected to be considerably less than that for the concrete strength, which had a standard deviation of about 300 lb. per square inch, caused by variation in water/cement ratio alone. The magnitude of the error emphasized the need to avoid the popular fallacy of equating the variation in concrete strength due to cement variation, to the *observed* variation of mortar cubes. It was also clearly advisable to base deductions on concrete tests, and not on mortar tests.

Dr Plowman's belief that statistics could be applied only to very large numbers of results was a fallacy, and for certain purposes fifty was a large, and perhaps unnecessarily large, number. Whatever the vagaries of concrete strength they were at any rate more predictable than public opinion. If they could be affected by a political scandal or a newspaper campaign the life of the civil engineer would indeed be a hard one. However, Dr Plowman's doubts did not prevent him from applying statistics to sets of 23 and 27 results. Whilst unable to follow the argument after Table 16, the Author agreed that testing machine errors might be important. Even more serious than the point mentioned by Dr Plowman was the considerable bias which could apparently exist between different machines.

Mr Neville's comments were interesting. The error in adding a predetermined amount of water would certainly not be proportional to the amount added, but might increase to some extent as the nominal amount increased. It appeared that the effect of the error was not greatly affected by the assumption made, when water was added to a given weight of cement to produce different water/cement ratios.

Paper No. 5958

"Guildhall: Provision of New Foundations to the North Wall by Underpinning and Other Means"†

by

Burnard Geen, M.I.C.E.

Correspondence

Mr J. W. Alexander complimented the Author on his novel method of underpinning and the City Corporation on adopting his ideas. The historic status of Guildhall classed it as a National Monument and the ingenuity and courage of the Author and of all those who had taken part in the constructional work had been well justified and proved by the test loading.

The observations made on p. 203 concerning the differences in the ground levels were interesting and strengthened the general belief that in Yellow or London Blue Clay particularly the water-table bore little relationship to the natural drainage. Many boreholes had been taken, both prior to and during the construction of Tilbury generating station on the banks of the Thames, and water had been encountered in the bores at almost any depth from ground level down to the ballast strata about 60 feet below high water level. That was attributable to the layers of peat in the clay at various depths through which the ground-water percolated. Where no peat bands had been encountered the bores had been quite dry until striking the ballast. Upon reaching the ballast the water had risen in the tubes almost to the top.

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† Proc. Instn Civ. Engrs, Part I, vol. 3, p. 201 (Mar. 1954).

Sudden changes of the ground-water level could also be temporarily, or occasionally permanently, caused during basement construction by drying out and continuous pumping or "well-drawing." Had either of those occurred at the Guildhall?

In view of the expense and difficulty under which the piling had been executed and the satisfactory nature of the test loading on buttress No. 2, Mr Alexander wondered why the pile capacity on subsequent buttresses had been restricted to 45 tons, especially with a 10-foot penetration into the Blue Clay. An arrangement of seven piles per buttress, giving a maximum load of 60 tons, with two piles on each side of the buttress and three piles along or under the wall, would appear to give a satisfactory and more economical arrangement.

The method of test-loading the piles was very compact and ingenious under the restricted headroom and limited working space, and strengthened the argument in favour of the hydraulic-jack system of test-loading of piles. The direct method was, however, more positive and gave a very constant load, but it did need "free" conditions. Direct loading would, however, have obviated the necessity of grouting in the flat jacks, and the factor of safety could have been practically obtained by loading up the whole system after construction, when the pile had taken up the dead load of the buttress wall—but perhaps that was too much to expect in modern times.

Mr S. A. Paris observed that no information was given in the Paper regarding the properties of the stratum of ballast (see p. 202). Whilst many foundations had in fact been taken down to the London Blue Clay he doubted if that was really necessary. If the ballast was of high density it was possible that, owing to its high angle of internal friction, it would support piles with a considerable load at depths of the order of 10 feet in that material. That would have the advantage of needing only short piles and of spreading the load evenly on to the top of the clay strata. By sinking the piles into the Blue Clay one could expect a greater settlement with the years.

Jacks were used to test the piles and Mr Paris suggested that it would have been possible, by having beams of a different design, to drive the piles from them by means of the jacks. In that way the positive resistance put up by the ballast would have been known for any depth and if insufficient the pile could have been driven farther, right into the Blue Clay.

The piles would probably have consisted of short precast concrete segments and in way of the beams cranked below the Crypt floor the piles might have been driven through openings in the beams, the jacks working against steel straps temporarily attached to them.

In *Figs 4* there was a note which called for only a nominal settlement of the pile under the load applied by the jacks. Was it not possible that the pile would have settled further if the loading had been maintained for a considerable period? No information was given regarding the length of time the pile was under the test load. Mr Paris felt that a plot of the time/settlement curve might have yielded interesting information.

Referring to the use of "dry" grout, he suggested that, whilst the contraction was necessarily small, it did exist. Perhaps in future work of the same nature an expanding-cement technique would be used to ensure the necessary stress in the underpinning piles. That had been accomplished at Lille, apparently quite successfully, in the underpinning of the Palais Rihour.[1]

Mr D. W. Cracknell observed that the Author had made very effective use of the basic ideas of prestressing, whereby separate parts of a beam were united in the prestressing operation thus enabling maximum advantage to be taken of the technique. If there were any doubts at all about the efficacy of the method adopted, they would be in respect of the loss of prestress which might result after a long period from inelastic deformation of the existing buttresses, which appeared to have been constructed in a manner likely to conduce to such movements.

The stress/strain characteristics of the prestressing wires were interesting. The modulus of elasticity obtained in the Author's tests (30,000,000 lb. per square inch) was much higher than Mr Cracknell had generally found. In some cases tests had revealed values as low as 24,000,000, although those exceptionally low values were considered to be indicative of piping in the wire, which resulted in a higher stress than that calculated. It seemed to be generally agreed that steel subjected to considerable cold working had a lower modulus of elasticity than that which had not been cold worked and a value for E of 28,000,000 lb. per square inch appeared to be satisfactory for high-quality wire. The Author's test results were therefore interesting, and information regarding the composition and manufacture of the wire would be appreciated.

Engineers in the London area would be grateful for the test data given by the Author for the bored piles. It was not often that arrangements could be made for testing that type of pile and every such test recorded helped to diminish the uncertainty which appeared to exist regarding the load-carrying capacity of the bored pile. Mr Cracknell concluded that all the piles had been pre-loaded by means of jacks, and the use of Freyssinet flat jacks for that purpose was interesting. The small size and relatively low cost of those jacks might encourage engineers to be more insistent on the simple, but usually very necessary, precaution of preloading underpinning piles or bases before the existing work was finally brought to bear thereon. Often the positioning of bored piles was important; could Mr Green say what accuracy had been achieved on the contract?

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[1] Henry Lossier, "L'autocontrainte des bétons par les ciments expansifs." ("Stressing of concrete by means of expanding cement.") Mem. Soc. Ing. Civ. Fr., vol. 101, p. 189 (Mar.–Apr. 1948).

It was to be hoped that further descriptions of the work of reconstruction at Guildhall would be made available.

Mr J. S. Wilson observed that the problem of providing additional support to an old structure without disturbing the old masonry was one which had defeated many engineers, and the basic features of the Author's special adaptation of prestressing was sure to be adopted in the future by many who had to deal with old structures.

The Author's solution to the problem presented by the want of head room and working space was also of great interest.

In treating cases of old structures needing consolidation or additional support, Mr Wilson had found it useful to investigate any possible settlement which might have taken place. He had found that quite substantial settlements were often evident, but tests had proved that they had all occurred possibly centuries earlier. Had the Author noticed any settlement of that nature in the structures he had dealt with?

Guildhall was the very centre of civic life in the Commonwealth, and the consolidation of the foundations of such a famous structure was a task with which any engineer would be proud to be entrusted.

Mr E. M. Hayward stated that he had been engaged in the earliest discussions concerning the various methods of underpinning Guildhall, and had later taken some part in the work.

The Author had stated that it had not been possible to make any reasonably accurate calculations regarding loads and ground pressures, nor to prepare a scheme until a complete survey had been made of the walls and piers. The architect had also required a detailed survey of the Hall, in order to consider the form of arch for the new roof which would best harmonize with the existing work. An engineer had spent about 6 months in measuring the smallest architectural details of the wall, and had produced more than sixty large drawings which had proved to be of great value in the preparation of the detailed drawings for the underpinning and of the architectural details of the new roof. At the same time, the positions of all cracks and badly calcined areas of stonework had been marked on the drawings. Some of the cracks in the buttresses were obviously of a recent nature and metal tell-tales had been fixed across those cracks. It was a remarkable tribute to the success of the underpinning operations that no movement had occurred in any of those tell-tales, a very satisfactory result which would hardly have occurred with any of the other more normal methods of underpinning.

In order to emphasize the extremely limited working space, Mr Hayward pointed out that the piers were at about 20-foot centres and roughly 5 feet wide, although they spread in some cases to about 7 feet. (See *Fig. 2*, facing p. 204.) That had left a narrow passage about 5 feet wide between the north faces of the piers and the adjacent wall of the temporary building, and not more than 13 to 15 feet between the pier faces. In several bays the clear space between the piers had been further limited by existing stairs, brick walls, and a waiters' lavatory. There had been only pedestrian access between the north and south faces of the wall. That access could not be used for plant movements or even barrows. All excavated material, piling plant, and concrete had had to be wheeled long distances down narrow passages, which had had to be kept free of obstructions at all times for the use of Guildhall staff. Guildhall Council Meetings, banquets, and other functions had continued to take place during the course of the work. In consequence, the noise from compressors, piling plant, and hammers had had to be restricted to certain times.

Such restrictions of space and time were part and parcel of a contractor's life. No mention need have been made of them, but for the fact that they had considerably affected the final decision as to the method of underpinning to be adopted.

The Author had mentioned that the existing buttresses, carrying about 400 tons each, were founded on poor Yellow Clay above the thick gravel bed. Excavations on the adjoining site had shown that a considerable volume of water was present in that gravel. If new foundations had been constructed by underpinning down to the gravel layer, then they would almost certainly have suffered further disturbance during the construction of the deep basement on the adjacent site. It had been essential, therefore, to carry those heavy loads down to the level of the Blue Clay at a depth of about 20 feet below floor level. That might have been done in the usual way by underpinning in stages with mass-concrete blocks. By that method, to allow sufficient working space, at least one-sixth of the foundation area would have been opened up, adjacent areas would have been disturbed during excavation, and heavy pumping would have been necessary. That had been considered too slow, costly, and dangerous to the safety of the old structure.

Another quite usual method would have been to pile on each side of the existing foundation and transfer the load to the new foundations by means of steel or reinforced-concrete "needles." Steel had still been licensed and difficult to obtain at that time; apart from that, heavy steel girders would have been difficult and in some bays impossible to place. Reinforced-concrete beams would have been easier to place than steel, but both of the needling schemes suffered from the same major disadvantage—that dangerously large areas of foundations would have been opened, and it was almost certain that there would have been settlement and cracking of the superstructure. Risks of that nature could not be taken with an old building of such historical value.

The scheme devised by the Author to overcome those difficulties was really ingenious in its simplicity, its effectiveness, and its adaptability to the varying conditions at each buttress. In effect, new concrete blocks had been knitted to the old foundation by means of prestressed cables and thus the foundation had been extended and the load transferred to the adjacent piles with no measurable settlement, nor at any time inviting disaster by considerably disturbing the existing yellow clay which had already been dangerously overloaded. After completion of the first buttress, which had been in the nature of a test to verify the effectiveness of the scheme, the remainder of the work had been completed remarkably quickly and cheaply.

Mr C. Djanoeff observed that the Author had not only opened a new chapter in methods of underpinning but had, by his application, provided ample food for thought on the other uses to which the principles and materials of prestressing and poststressing could be put.

It would appear that the structure had contributed to the general success of the method adopted by the uniform soundness of the material of its buttresses.

The Paper might provide an opportunity of bringing to the notice of those concerned with underpinning problems the existence of a French controlled-expansion cement, which was manufactured and could be obtained in France. The quantities required for the last phases of underpinning work were generally small and might in certain cases justify its use, despite its cost.

Mr G. C. Wilson observed that, in a problem which presented many variables, the only factor which appeared to have been at all uniform was the nature of the ground below the foundations. The work had provided a further example of the specialized use of bored piles, and emphasized their advantages where working space was very limited and where it was necessary for disturbance due to shock and vibration to be reduced to the minimum. The combination of that type of pile, which was readily constructed to the length required, with the " vice-jaw " beams formed a very flexible system which had been neatly adapted as the circumstances demanded.

The survey of the north wall had been made very thoroughly, and had included an examination of the solidity of the buttresses and the testing of the material used most extensively in their construction. On the other hand, the site exploration had been limited to the determination of the soil profile and the testing of the ground-water. He suggested that the measurement of soil properties, primarily the shear strengths of the clays, would have been a valuable addition to the investigation and would have provided data required later for the design of the piles. In the absence of such information, how had the size of pile been selected ?

Another advantage of the bored pile technique was that a constant record could be obtained of the soils into which the pile was penetrating. Not only was the type of soil indicated, but " undisturbed " samples could be taken suitable for strength determinations. If the soils were clays, such as the lower strata beneath Guildhall, they could be tested on site to give a measurement of the shear strength of the soil. Those values could then be used to check if the pile had entered a sufficiently strong stratum for an adequate depth, that was to say, if the pile would be able to develop its required bearing capacity. Had any such site measurements been made during the installation of the piles ?

Referring to the lengths of the piles, the Author had stated that in the underpinning of buttress No. 2, which had been considered as pilot experimental construction, the piles had been bored 5 to 8 feet into the stiff Blue Clay, leaving the length to be adopted for the remainder of the work to be decided on the basis of the results of loading tests. Those results were quoted, from which it could be inferred that the test piles had been satisfactory. It had been decided, however, that the piles should penetrate 10 feet into the Blue Clay. Would the Author explain why the additional length of pile had been considered necessary ?

Preloading of piles was a procedure to be commended, but where the foundation loads were to be supported by clay soils consolidation settlement would occur over a long period of time. To reduce substantially subsequent movement of the structure, it was therefore necessary to keep the proof load operative for an appreciable time. No details of the duration of the loading tests were given, but if the loads had been applied for only short periods the measured movements would have been largely the direct compressions of the piles. Such tests could not therefore give a reliable guide to the eventual settlement of the structure, nor could they indicate the load-carrying capacities of the piles. Mr Wilson felt that the evaluation of the test piles might have been based on unsound premises. Further information from the Author about the tests would be welcomed.

Lastly, with reference to the second conclusion drawn from the pile tests, had any measurements been made of relative movements between the beams and the buttress ? If not, on what observations had the conclusion been based ?

The Author, in reply, referred to Mr Alexander's question on the variation of ground-water level. No doubt during adjoining basement construction about to be undertaken, where considerable pumping would be required, some change was probable—either temporary or permanent.

Limiting the load on the piles to about 45 tons was a question of mental approach. It had been necessary to arrange beforehand some comprehensive plan ; since, had a higher figure been assumed than could subsequently be justified, a very serious situation could have arisen, and it should be remembered also that it had been necessary to proceed with all possible dispatch.

It had become, therefore, very much a question of individual judgement and, from a long experience of the use of such piles and of many loading tests, it had been the Author's opinion that "elbow room" would be necessary in fixing a safe capacity.

The question of direct test-loading had been considered but, owing to the extremely limited access and space, it had been considered less suitable than the method of pre-loading adopted.

Mr Cracknell had raised the interesting point of the modulus of elasticity in the tested samples of wire, but there did not seem to be any good reason for doubting the accuracy of the tests made or that they were representative of the material employed.

The question of loss of prestress which might result after a long period was a pertinent comment to which, on present information, there did not appear to be a certain answer. It was felt that a remedy was possible, if ever required in the future, by the simple expedient of redrilling and inserting new cables in new positions.

With regard to the positioning of the piles, it had been a simple matter to retain their position relative to the side of the rigid buttress and any slight error in the opposite direction along the beam was unimportant.

Mr J. S. Wilson had asked whether any substantial settlements which might have occurred many years previously had been noted; many indications of such settlements had been observed, and it was the Author's opinion that the level at which the underside of the buttresses at present sat was very considerably lower than when first built.

The subsoil had been squeezed and its density increased so that the ground pressure existing before provision of the new foundations must have been of the order of from 5 to 6 tons per square foot.

The Author was grateful to Mr Hayward for having set out at some length the operational difficulties arising from the very limited access and space available and other matters which had been omitted from the Paper for the sake of brevity. .

Mr Djanoeff had referred to the use of controlled-expansion cement, the use of which in some of the "closing" operations might have been an advantage; but since it had appeared likely to take some considerable time to obtain such cement and study its use, it had not been employed and hand-driven packing had been used instead, in which the small depth had not been expected to produce sufficient shrinkage to matter.

Mr G. C. Wilson had raised the question of duration of pile tests and it was accepted that the testing carried out had been in no sense a complete answer to the sufficiency or otherwise of the piles provided. That was one of the reasons why a pile of abundant capacity had been employed and a length adopted which might in fact have been a little more than necessary. However, considering the small ratio which the cost of such additional length of pile or even diameter of pile bore to the whole cost of the operation, it seemed to the Author that theoretical considerations were not the most important; the whole matter should be subject to personal judgement.

Prolonging the pile tests to a reasonable period had been quite out of the question and it had been necessary to proceed with all the speed which could be obtained, in the light of the fact that movements were taking place. It was the Author's experience that unless, at a test load of 50 tons, a 19-inch pile showed considerable and increasing movement, it would carry 60 tons under longer exposure to the test load and that figure had seemed under all the circumstances to be a reasonable margin to allow.

No measurement had been made to determine if there were any relative movements between beam and buttress, because it had been felt that there would be immediate visual demonstration if such had occurred.

It was perhaps interesting to observe that since writing the Paper a portion of the new roof had been completed and some of the foundations fully loaded.

Mr Paris had referred to the possibility of using short piles finishing in the ballast, which, on an open site without other conflicting factors, might have provided the support required. In the case under discussion it had been considered that the necessity to construct a deep basement close to the new work with its contingent pumping, as referred to by Mr Hayward, rendered such an idea quite unsafe. The Author agreed, however, that cases did arise where the idea of short piles in a firm ballast was the correct answer and had in mind a case with which he had dealt 30 years previously, where by driving many hundreds of short piles it had been possible to avoid working in water.

Mr Paris had suggested an alternative method of reaction piles; it was fair to say that many alternative methods suggested themselves and it was not claimed that the method adopted was necessarily the only one which would have produced the required result. His useful mention of expanding cement had been answered in the reply to Mr Djanoeff, and he was probably right in suggesting that it would be extensively used in the future.

ELECTION OF ASSOCIATE MEMBERS

The Council at their meeting on the 21st September, 1954, in accordance with By-law 14, declared that the undermentioned have been duly elected as Associate Members:

Ackers, Godfrey Lloyd, M.A. (*Oxon.*), Grad.I.C.E.
Allan, Henry Kenneth Royston, B.Sc. (*St Andrews*), Grad.I.C.E.
Allison, Alexander Morgan, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Archer, Maurice Frederick George, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Bartholomeusz, Christopher Eric, Stud.I.C.E.
Beca, George Samuel, B.E. (*New Zealand*).
Bedson, Peter George, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Birse, Ronald Macdonald, B.Sc. (*Edinburgh*), Grad.I.C.E.
Bishop, Donald.
Braatvedt, Ian Harold, B.Sc. (*Witwatersrand*).
Bradley, Peter George, B.Sc. (*Cape Town*).
Bright, Wilfred Francis Laurie, B.Sc. (*Witwatersrand*).
Buchan, Ian, B.Sc. (*Glasgow*).
Burger, Bernardus Antonius, B.Sc. (*Cape Town*).
Burrow, John Cummings, B.Sc. (*Cape Town*).
Burt, Robert Cooper Burgon, B.Sc. (*Edinburgh*), Grad.I.C.E.
Carter, Anthony Leslie, B.Sc.(Eng.) (*London*).
Chapman, Henry Bryan Parry, B.A. (*Cantab*), Grad.I.C.E.
Chapman, Robert George, B.C.E. (*Melbourne*).
Chilton, William Gordon Eley, M.A. (*Cantab.*), Stud.I.C.E.
Collins, Anthony Martin.
Coney, Harold Edward, B.Sc. (*Witwatersrand*).
Cox, Cecil Gordon, B.A. (*Cantab.*).
Cumming, Alastair Paul, B.Sc.(Eng.) (*London*).
Cunnington, Frederick Peter, Grad. I.C.E.
Dall, Ewart Montgomery, B.Sc. (*Witwatersrand*).
Davie, John (*Jnr*), B.Sc. (*Glasgow*), Grad.I.C.E.
Dean, Noel, Grad.I.C.E.
Dencer, Geoffrey Hargreaves, B.Sc. (*Manchester*), Grad.I.C.E.
de Waal, Frans Ignatius, M.Sc. (*Witwatersrand*).
Dickson, Arthur, Grad.I.C.E.
Donald, Allan, B.Sc. (*Glasgow*).
Duke-Cohan, Elias Marcus, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Eaton, Alexander Dennis, B.A., B.A.I. (*Dublin*), Grad.I.C.E.
Edwards, William Howard, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Eveleigh, William James Geoffrey, Grad.I.C.E.
Falconer, Bruce Halcro, B.E. (*New Zealand*).
Faure, Peter David, B.Sc. (*Cape Town*), Grad.I.C.E.
Geatrell, Donald Ernest, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Gibb, Oswald, B.Eng. (*Liverpool*), Grad.I.C.E.
Gillis, Paul, B.Sc. (*Cape Town*).
Giddy, David Hoskins, B.Sc. (*Witwatersrand*), Grad.I.C.E.
Gillison, Thomas Clive, Grad.I.C.E.
Graham, Robert Dickson, B.Sc. (*Edinburgh*), Grad.I.C.E.
Granger, Neville Aloysius, B.Sc. (*South Africa*).
Gray, Alfred Michael Bentley, B.Sc.(Eng.) (*London*).
Gray, John Campbell, M.Sc.(Eng.). (*London*), Stud.I.C.E.
Griffiths, William Carmichael, B.Sc. (*Cape Town*).
Gross, Michael, B.Sc. (*Cape Town*).
Hailey, John Sealy Marsh, B.A. (*Cantab.*), Grad.I.C.E.
Halsall, Robert, B.Sc. (*Glasgow*), Stud.I.C.E.
Handley, Walter Dixon, Grad.I.C.E.
Hargreaves, Robert Adrian.
Hawkes, Jeffrey, B.Sc.(Eng.) (*London*).
Henderson, Robert Waldemar, B.Sc. (Eng.) (*London*).
Henny, Gerhardus Evert Jan, B.Sc. (*Witwatersrand*).
Herd, Donald Wallis, B.E. (*New Zealand*).

Heywood, Peter Robellaz, B.Sc. (*Witwatersrand*).
Hickfang, Jack Millwood, B.Sc. (*Cape Town*).
Houston, Thomas Ronald, B.Sc. (*Belfast*), Grad.I.C.E.
Howard, John Meredith, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hubbard, Stanley Albert, B.Sc. (*Witwatersrand*).
Hughes, Derrick Hurdis, B.Sc. (*Bristol*).
Irving, Derek Arthur, Grad.I.C.E.
Jones, Malcolm Leslie, B.E. (*New Zealand*).
Jones, Richard Robert Ross.
Katz, Louis, B.Sc. (*Witwatersrand*).
Key, David Edwin, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Kimelman, Richard, B.Sc.(Eng.) (*London*).
Kinnear, Robert Ian, B.Sc. (*Edinburgh*), Grad.I.C.E.
Kinnes, George Carr, B.Sc. (*St Andrews*).
Langerman, Peter Stuckeris, B.Sc. (*Witwatersrand*).
Legge, William Clement Sinclair, B.Sc. (*Witwatersrand*).
Littlewort, Charles James, B.Sc. (*Cape Town*).
Llewellin, John David, B.A. (*Cantab.*).
Lowther, William Arthur, B.Sc. (*Durham*).
McCaffrey, Dermot William, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
McCaughan, Harold Thompson, B.Sc. (*Belfast*), Grad.I.C.E.
McKee, Maurice Edgar, B.Sc. (*Belfast*), Grad.I.C.E.
McKenzie, Gavin Wilson, B.Sc. (*Glasgow*), Stud.I.C.E.
McKie, Donald Lancelot.
MacLean, Richard Fredrik, B.Sc. (*South Africa*).
McQueen, Maurice Fraser Spence.
Mason, Anthony Frederick Festing.
Mason, Vernon Davis, B.Sc. (*South Africa*).
Maxwell, Robert Morell, B.Sc. (*Belfast*).
Meigh, Alan Cotton, M.Sc.(Eng.) (*London*).
Melvin, Colin Gordon, B.Sc.(Eng.) (*London*).
Milner, Thomas Gordon, B.Sc. (*Aberdeen*).
Mitchell, James.
Morham, Malcolm Dunbar, B.A. (*Cantab.*), Stud.I.C.E.
Moulden, Frank James, Grad.I.C.E.
Neville, Adam Matthew, M.Sc.(Eng.) (*London*).
Nicol, John Keith James, B.Sc. (*Natal*).
Niebieszczanski, Wieslaw Andrzej.
Norton, Henry George William.
Parton, Girvan Meir, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Patkunarajan, Navaretnam, Grad. I.C.E.
Peart, Gerald Ernest Walter, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Pickford, John Aston, B.Sc.(Eng.) (*London*).
Pickup, David Peter, B.Sc. (*Durham*), Grad.I.C.E.
Pollard, Eric Stephen, B.A. (*Oxon.*).
Price, Jack, Grad.I.C.E.
Putterill, Kenneth Edgar Charles.
Pyne, David Stanley.
Ramsey, Frank Cyril.
Rendle, Philip John, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Robinson, Geoffrey William, B.A. (*Oxon.*), Grad.I.C.E.
Robison, Walter Frederick, Grad. I.C.E.
Rocke, George, B.Sc. (*Glasgow*), Stud. I.C.E.
Roydhouse, Peter Whitton, Stud. I.C.E.
Russell, George Edward.
Savage, Christopher David Wyndham, B.A. (*Cantab.*), Stud.I.C.E.
Schneider, Saul Jacob, B.Sc. (*Cape Town*), Grad.I.C.E.
Schofield, Lewis Arnold, B.Sc. (*Leeds*).
Senior, Edward, Grad.I.C.E.
Shand, Ian Harry Robertson, B.Sc. (*Cape Town*).
Sheach, Lawrence James, Grad.I.C.E.
Simpson, William Rattray, M.A. (*Cantab.*).
Smith, Gerald Lupton, B.Sc. (*Glasgow*).
Smith, Iain Parker, B.Sc. (*St Andrews*), Grad.I.C.E.
Smith, Reginald George, B.Sc. (*Edinburgh*), Stud.I.C.E.
Smith, Wilfred Collier, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Spottiswoode, Robert Anthony, B.A. (*Cantab.*).
Stephens, George Raymond, Grad. I.C.E.
Still, Charles Albert, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Tootill, Albert, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Townend, Alan Dennett, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Truebody, Charles Thomas, B.Sc. (*Cape Town*), Grad.I.C.E.
van Gemert, Bastiaan, B.Sc. (*Cape Town*).
Walecki, Witold Andrew, B.E. (*National*), Stud.I.C.E.
Waring, Alan, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Warner, Colin Arthur, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Webster, John Alexander, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Wells, Derek Frank, B.Sc. (*Cape Town*).
Welsby, Alan, B.Sc.Tech. (*Manchester*).
Whitelock-Jones, Christopher John, B.Sc. (*Witwatersrand*).
Widdowson, Peter Wright, B.Sc. (*Manchester*), Stud.I.C.E.
Wideman, Robert Max, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Wijesinghe, Thilakaratne, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Wills, Dudley Spencer, Stud.I.C.E.
Wilson, David Murray, B.E. (*New Zealand*), Stud.I.C.E.
Wilson, Gordon, B.Sc.(Eng.) (*London*).
Woodman, Peter Edward, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Wright, Colin John, B.Sc.(Eng.) (*London*), Stud.I.C.E.

and the following as Associates :

Francis, John Robert Dark, B.Sc. (Eng.) (*London*).
Morton, Edgar, M.Sc.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

Robert Baker, O.B.E. (E. 1915).
Thomas Bell, E.D. (E. 1927, T. 1946).
Ernest Andrew Borg (E. 1935).
William Cecil Clemens, O.B.E. (E. 1912, T. 1938).
Joseph Henry Dobson, D.S.O., D.Eng., D.Sc. (E. 1904, T. 1919).
John Leonard Eve (E. 1938).
Robert Charles Frain (E. 1920).
Harold Moffat Swire Haughton, B.Sc. (E. 1910, T. 1933).
David Henderson (E. 1910, T. 1933).
Herbert Ross Hooper, O.B.E., M.A. (E. 1890, T. 1903).
Ethelbert Monk Newell, D.S.O. (E. 1918, T. 1929).
William Duncan Lund Roberts (E. 1907, T. 1927).

Associate Members

William Butterworth (E. 1926).
Clement Frederick Davis, B.A. (E. 1897).
Arthur Henry Johnstone, B.A.I. (E. 1898).
John Grove Stanley Middleton, B.Sc. (E. 1916).
William Morris Prout (E. 1896).
Frank James Smith, M.B.E. (E. 1934).
Sir Herbert (Geraint) Williams, Bart., M.P., M.Sc., M.Eng. (E. 1910).
Charles Edward Woodger, B.Sc.(Eng.) (E. 1928).

Student

Peter Edward Woodman, B.Sc.(Eng.) (A. 1947).

OBITUARY

WILLIAM CECIL CLEMENS, O.B.E., who was born on the 2nd August, 1882, died at the age of 70 at Mevagissey, Cornwall, on the 18th November, 1952. He was educated at Stoke Naval and Civil Service College, and at Devonport High School, and received his early practical training (1897–1902) under Mr A. W. Debnam, Surveyor to East Stonehouse U.D.C.

His first practical experience (1902–1906) was obtained as Engineering Assistant to a firm of contractors and later (1906–1909) to Mr J. F. Burns, Borough Engineer of Devonport, and consisted of the initial surveys, preparation of plans, and estimates for street works, road improvements, drainage schemes, housing estates, and public buildings.

In 1910 he was appointed Chief Engineering Assistant to Mr A. E. Derby, Borough Engineer, Bethnal Green, and until 1915 was engaged upon general municipal engineering works.

After the 1914–18 war (during which he served as Engineering Inspector to the Road Board and Joint Roads Committee) he joined the engineering staff of the Roads Department, Ministry of Transport. In 1923 he was appointed Deputy to Mr A. J. Lyddon, Chief Engineer, Arterial Roads, and was engaged upon the construction of 210 miles of arterial roads in the Greater London area at a cost of £14,000,000. In 1928 he succeeded Mr Lyddon as Chief Engineer.

From 1929 to 1936, Mr Clemens became Engineering Inspector in charge of general engineering under Major F. C. Cook, the Chief Engineer. He was then responsible for the examination of schemes, policy, specifications, and standard conditions of contract for trunk road programmes. He also did a great deal of work at R.A.F. stations on behalf of the Air Ministry. In 1936, he was promoted to Senior Engineering Inspector and later was connected with Public Inquiries held under the Public Works Facilities Act, 1930, the Road Traffic Acts, and the Restriction of Ribbon Development Act, 1935.

In 1938, Mr Clemens was appointed Engineer in Charge of Trunk Roads in connexion with the maintenance and improvement of 4,500 miles of public highway, which became the responsibility of the Minister of Transport on the 1st April, 1937, under the Trunk Roads Act.

During the 1939–45 war he continued to serve with the Ministry of Transport on work of national importance.

He was elected an Associate Member of the Institution in 1912 and was transferred to the class of Member in 1938.

JOSEPH HENRY DOBSON, D.S.O., D.Eng., D.Sc., M.Sc., who was born at Crewe in 1878, died at his home in Johannesburg on the 29th June, 1954.

Before commencing a distinguished career in academic and practical engineering, he served a seven-year apprenticeship in the London and North Western Railway Company's workshops in Crewe. Then scholarships took him through Victoria University, Manchester, and Liverpool University, and in 1904 he was appointed Professor of Electrotechnics at the Transvaal University College (now the Witwatersrand University).

In 1911 he was appointed General Manager of the Johannesburg Gas, Electricity, and Tramways Department. During the 1914–18 war he served as Senior Engineering Officer to the Second South African Division. In the East African campaign, General Smuts decorated him in the field with the Distinguished Service Order.

In 1920, he acquired an interest in the engineering business of the late Dr R. M. Dowson. The company changed its name accordingly and for Dr Dobson it was the start of a career which played a major part in the industrial development of South Africa.

In 1926 he found time to take the degree of Doctor of Engineering at Liverpool University, and wrote the first of a series of twenty-one technical Papers which were to win for him the highest awards from a number of scientific societies. In 1929 he was elected Fellow of the Royal Society of South Africa.

From 1933 to 1935 he temporarily withdrew from his own business at the Government's request and took over the management of the Union Steel Corporation at a critical time when the Corporation was approaching production and had suffered the loss, by death, of a key man in the person of the works manager. On his return to Dowson and Dobson Ltd, he took an active interest in many other new industries.

Apart from membership of the Institutions of Civil Engineering, Mechanical Engineering, Electrical Engineering, and Chemical Engineering in London, Dr Dobson's academic qualifications were exceptional. He was twice awarded the Gold Medal of the South African Institute of Engineers, and was also a Gold Medallist of the Institutions of Electrical Engineers and of Chemical Engineers, London. He was a former President of the South African Institute of Engineers, and of the South African Electrical, Chemical, and Mining Society, and of the Associated Scientific and Technical Societies of South Africa. In 1946 the University of the Witwatersrand conferred on him the Honorary Degree of Doctor of Science in engineering.

He was elected an Associate Member of the Institution of Civil Engineers in 1904 and was transferred to the class of Member in 1919.

Dr Dobson, whose wife died in 1948, is survived by two daughters.

GEORGE McILDOWIE, who was born at Strandtown, Belfast, in 1886, died in Cobham Hospital, Surrey, on the 28th December, 1953. He was educated at Sedbergh School and gained his early practical training under Mr B. D. Wise and Mr B. Malcolm (1904–07) and later (1907–09) as an assistant in the Permanent Way Department of the Midland Railway, Belfast.

From 1909 to 1911 he studied engineering at Queen's University, Belfast, and then took up a position as Senior Assistant on the surveys, plans, and estimates for the Silent Valley Storage Reservoir (under Mr F. W. McCullough). This work occupied him until 1922 except for a period of war service with the Royal Engineers.

From 1923 to 1933 he was the Resident Engineer on the construction of the reservoir, and in 1935 presented to the Institution a Paper[1] on that work, for which he was awarded a Trevithick Premium. From 1934 to 1937 he was Resident Engineer in charge of the duplication of the pipe section of the Mourne Conduit (Belfast Water Supply).

In 1938, he joined the staff of the Metropolitan Water Board and was appointed Resident Engineer for the construction of Walton Reservoir. After the war he was concerned with the large post-war programme of new works. At the time of his death he held the appointment of Deputy New Works Engineer of the Metropolitan Water Board.

He was elected an Associate Member of the Institution in 1913 and was transferred to the class of Member in 1936. He served on the Panel of Civil Engineers qualified under the Reservoirs (Safety-Provisions) Act, 1930, to design and construct all forms of reservoirs.

Mr McIldowie is survived by his wife and one son.

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[1] "The Construction of the Silent Valley Reservoir, Belfast Water-Supply." Min. Proc. Instn Civ. Engrs, vol. 239 (1934–35, Pt I), p. 465.